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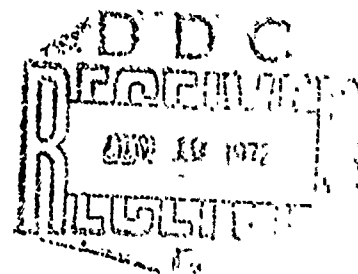


## USAF MOBILITY PROGRAM WASTEWATER TREATMENT SYSTEM

V. L. Snoeyink, et al.  
University of Illinois

TECHNICAL REPORT NO. AFWL-TR-71-169

April 1972



**AIR FORCE WEAPONS LABORATORY**  
Air Force Systems Command  
Kirtland Air Force Base  
New Mexico

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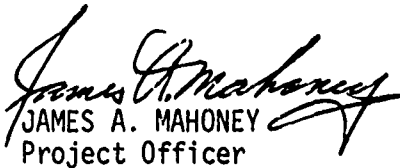
FOREWORD

This report was prepared by the University of Illinois, Urbana, Illinois, under Contract F29601-70-C-0087. The research was performed under Program Element 64708F, Project 3783, Subtask 3.6010.

Inclusive dates of research were 19 June 1970 through 18 February 1972. The report was submitted 13 March 1972 by the Air Force Weapons Laboratory Project Officer, Mr. James A. Mahoney (DEE).

The organization for this study consisted of two groups. The first group included the principal investigator, Dr. V. L. Snoyink, and F. I. Markus, B. S. Shin, and J. C. Lowe, graduate research assistants. The second group served primarily in an advisory capacity and was comprised of Drs. R. S. Engelbrecht, J. T. Pfeffer, and R. I. Dick. Periodic meetings were held between the two groups to obtain advice and information on various aspects of the investigation from the advisory group.

This technical report has been reviewed and is approved.

  
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ABSTRACT

(Distribution Limitation Statement A)

The U. S. Air Force Bare Base Mobility Program involves a highly mobile force of 1000 to 6000 men who can be moved any place in the world on a very short notice. The support systems for this force include a wastewater treatment system which can treat wastewaters to the required degree prior to discharge to the environment. The wastewaters which are generated at a bare base include photographic, aircraft and vehicle washrack, human, shower and lavatory, hospital, dining room, kitchen and laundry wastewaters. A waste treatment system which involves 1) separate collection and incineration of human waste, and 2) treatment of all wastewaters except concentrated photographic wastes in a system which includes chemical clarification, flotation, filtration, activated carbon adsorption and chlorination is recommended. The sludge, concentrated photographic waste and the skimmings from the aircraft and vehicle washrack wastes are incinerated and the ash from the incinerator is disposed of on land. A reclamation system consisting primarily of a reverse osmosis process is recommended for upgrading the quality of the effluent from the waste treatment system such that it is suitable for reuse. The brine from the reverse osmosis treatment is disposed of either by dilution in receiving waters, evaporation from ponds, or by transportation from the site.

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## SECTION I

### INTRODUCTION

#### 1. OBJECTIVE

The objective of this study was to investigate systems for collection, treatment and reclamation of wastewater, waste treatment residue disposal and effluent disposal, and to make recommendations for the most advantageous system or systems for use in the Air Force Bare Base Mobility Program.

The objective of the Mobility Program is quick worldwide deployment of a tactical force and all supporting facilities to a suitable airfield almost anywhere in the world. The number of personnel comprising this tactical force is 1000 to 6000 men in increments of 1000. Since the possible deployment sites include foreign countries as well as the U.S., various conditions for construction, pollution control and site restoration upon redeployment may be included in the occupational agreement. Some of the Bare Base sites may have to be located in arid regions where reclamation of water for reuse is an important consideration.

#### 2. CONSTRAINTS

The following constraints have been placed on the wastewater system in order that it be compatible with the Mobility Program:

- 1) The system must be capable of efficient operation almost anywhere in the world. A design range for temperature of  $-25^{\circ}\text{F}$  to  $+125^{\circ}\text{F}$  has been placed on the system. In addition, relative humidity and amount of precipitation may vary over wide ranges.

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- 2) The system must be fully operational within 72 hours.

Acceptable reductions in biochemical oxygen demand (BOD), suspended solids concentration and other pollutants must be achieved within this time limit. Alternatively, provisions must be made to satisfactorily contain the wastewaters until the treatment system becomes fully effective.

- 3) The system must incorporate satisfactory waste treatment residue handling and disposal facilities in order to prevent pollution of the countryside. Ultimate disposal of sludge is of particular importance when complete restoration of the site is required.

- 4) The system must be air-transportable. The C-130 aircraft is the smallest cargo-carrying aircraft utilized for such purposes. The critical envelope for the C-130 aircraft is 3 each at 8 ft x 8 ft x 12 1/2 ft, and the total cargo weight must not exceed 28,000 lbs. Envelopes are to be compatible with the 463L loading system; certain pieces of equipment may be lifted by helicopter.

- 5) The system must be fully operable on terrain which may vary from flat lands to hills and mountains and from swamps to land covered with ice and snow.

- 6) Ease of construction and dismantling of the system and simplicity of operation and maintenance must be considered in the selection of the best system. Much of the work involved in erecting, operating and dismantling the system may have to be done by untrained or semi-trained personnel.

- 7) The system must have a minimum of 5 years of operational life with two deployments per year. In addition, components should withstand a 10-year storage with appropriate inspection and maintenance (Ref. 1).

- 8) The wastewater treatment system must remove 90 percent or better of the BOD.

9) The reclamation system must produce water which meets the 1962 U. S. Public Health Service Drinking Water Standards.

10) The site must be readily restored to its original condition upon redeployment of the tactical force.

11) The requirements for chemicals, replacement parts and other supplies must be minimized.

12) Cost must be minimized.

### 3. METHODOLOGY

The specific procedures which were followed in the course of this study were previously presented in detail (Ref. 106). In summary, primary sources of wastewater and process data included various military agencies, equipment manufacturers and the technical literature. The base layout which was used to develop the treatment system is termed the "dispersed base" layout (see Fig. 1). Discussions with the planning division for the Bare Base Mobility Program at Langley Air Force Base, Virginia confirmed that this was indeed the layout which will be used.

The best means of treatment for the wastewaters was then determined on a source-by-source basis. Waste sources in close proximity to each other, such as shower and lavatory wastes from the housing units, kitchen wastes, laundry wastes and hospital wastes other than human wastes, were considered as one source because of relative similarities. Wastewaters such as the washrack wastes, the photographic wastes and the human wastes were considered separately to determine whether separate treatment or treatment with the other wastes was desirable. Factors entering into this analysis included the distance of transport, simplicity of process operation and minimization of

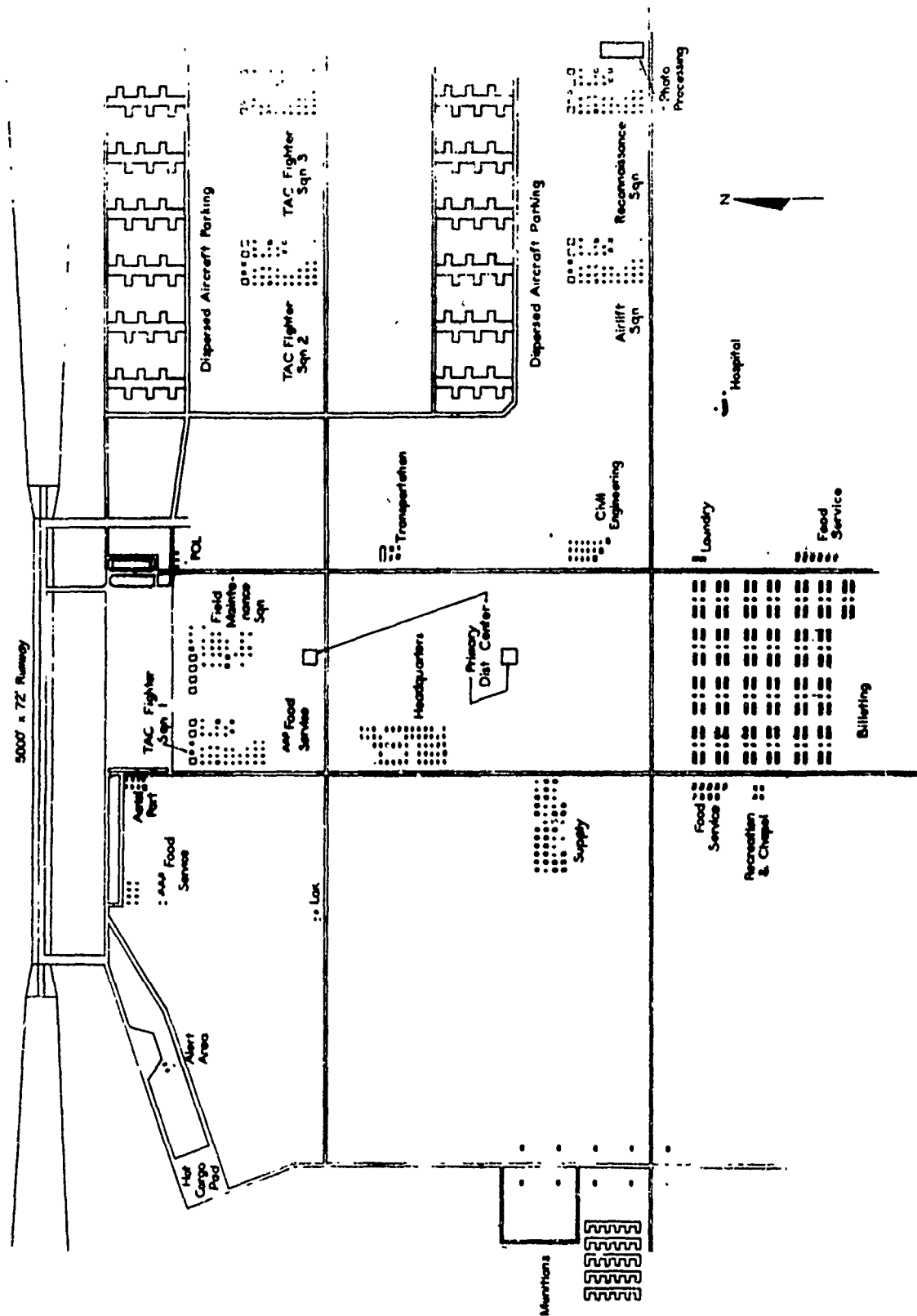


Figure 1. Dispersed Base Layout

manpower, minimization of operational difficulties, minimization of vulnerability to attack, probable effluent quality, potential reclamation of effluent, effect of mixing the waste with other wastewaters on the efficiency of the treatment processes being considered and cost of treatment.

The preliminary design presented in this report was based on the assumption that the treatment system will be developed in 1000-man units. Multiple units for bases larger than 1000 men will be located side-by-side for ease of operation. Development of the system in 1000-man modules was adopted primarily for ease of handling the packaged unit in warehouses, loading of the unit onto aircraft and uniformity in operation at the base site.

## SECTION II

### WASTEWATER SOURCES AND QUANTITIES

#### 1. SOURCES OF WASTEWATER

It was determined during the course of study that no major overhaul maintenance or repair capabilities for aircraft are planned for the Bare Base. On this basis, it is anticipated that the wastewater sources will be limited to 1) domestic wastewaters, including lavatory, shower, kitchen, dining hall and human wastes, 2) hospital wastewaters, including lavatory, shower, kitchen, dining room, human and other minor wastes and 3) industrial wastewaters, including photographic, laundry, aircraft washrack and vehicle washrack wastewaters.

#### 2. QUANTITIES OF WASTEWATER

The quantity of wastewater was determined by consideration of each waste source. In many cases it was necessary to make assumptions concerning the quantity of flow because of a lack of data.

The quantity of photographic wastewater was particularly difficult to determine. Based primarily on information given in Reference 8, it was assumed that:

1) A bare base, regardless of size, will have on line one Kodak Ektachrome RT continuous photographic processor, Model 1411-M, using the EA-4 chemical process. This is a mobile unit and can be readily deployed from base to base depending upon the operation mission.

2) No other photographic processing or drying equipment will be used on a bare base.

3) On a 1000-man bare base, the said photographic processing unit will be operational on a continuous basis 4 hours per day. At other times

the unit is assumed to be idle. The processing unit will be operated an additional 4 hours for each increment of 1000 men up to 6000 men. On this basis, 1600 gpd of photographic wastewater is produced for each 1000-man increment.

The quantity of aircraft washrack wastewater was calculated based on the assumption that a 6000-man base requires a 30 gpm pump to be operated 10 hours/day to supply sufficient water for washing aircraft (Ref. 7). The pump is operated proportionally less time for smaller bases. On this basis, a wastewater flow of 3000 gpd is produced for each 1000-man increment.

Similarly, the quantity of vehicle washrack waste was calculated by assuming that a 6000-man base requires a 10 gpm pump operating 10 hours/day to supply sufficient water for washing vehicles (Ref. 7). On this basis, a wastewater flow of 1000 gpd is produced for each 1000-man increment.

The quantity of hospital wastewater was calculated by assuming a 10-bed hospital for each 1000 men (Ref. 7). Assuming further that 50 gpd of wastewater is produced per bed, 500 gpd of wastewater is produced for each 1000-man increment.

The quantity of human waste (feces, urine, flush water, bactericide, etc.) was estimated in accordance with the following assumptions:

- 1) Human wastes will be collected and treated or disposed of separately from other wastewater streams.

- 2) Recirculating toilets similar to those used on commercial aircraft will be utilized. This arrangement minimizes both the amount of water supply required for the toilets and quantity of wastewater to be disposed of.

3) The per capita contribution to the human wastewater stream is based on values inferred from Garrett Airesearch Corporation's publications (Refs. 5, 6) where it is assumed that 80 to 120 men contribute 38 to 43 gpd or 0.31 to 0.55 gpd/capita. A figure of 0.55 gpd/capita was used to estimate human waste quantities.

The quantity of shower and lavatory wastewater was calculated by assuming 37 LUST units for a 6000-man base (Ref. 7). According to Reference 7, a flow of 1500 gpd/unit is expected. On this basis, 10 gpd/capita is produced. This value appears to be too low in view of the fact that 20 to 30 gals of water is required per shower (Ref. 107). Accordingly, a flow of 20 gpd/capita is assumed. On this basis, a 1000-man base will produce 20,000 gpd of shower and lavatory wastewater.

The quantity of kitchen wastewater was calculated by assuming that 14 kitchens with a waste flow of 600 gpd/kitchen would be used for 6000 men (Ref. 7), and with proportionally fewer kitchens for smaller bases. Accordingly, a 1000-man base is expected to produce 1400 gpd of wastewater.

The quantity of dining room wastewater was calculated by assuming that 14 dining halls with a wastewater flow of 250 gpd/dining hall are required for 6000 men (Ref. 7), with proportionally fewer dining halls required for smaller bases. The dining room waste flow is then 600 gpd/1000-man increment.

The quantity of laundry wastewater was determined by assuming 1 laundry unit/1000 men with a flow of 5500 gpd (Ref. 7).

The wastewater flows are summarized in Table 1.



TABLE 1  
WASTEWATER QUANTITIES

| Waste Stream                 | Base Size<br>(no. of personnel) |        |         |         |         |         |
|------------------------------|---------------------------------|--------|---------|---------|---------|---------|
|                              | 1000                            | 2000   | 3000    | 4000    | 5000    | 6000    |
| Photographic                 | 1600                            | 3200   | 4800    | 6400    | 8000    | 9600    |
| Aircraft Washrack            | 3000                            | 6000   | 9000    | 12,000  | 15,000  | 18,000  |
| Vehicle Washrack             | 1000                            | 2000   | 3000    | 4000    | 5000    | 6000    |
| Hospital                     | 500                             | 1000   | 1500    | 2000    | 2500    | 3000    |
| Human Waste                  | 550                             | 1100   | 1650    | 2200    | 2750    | 3300    |
| Lavatory and Shower          | 20,000                          | 40,000 | 60,000  | 80,000  | 100,000 | 120,000 |
| Kitchen                      | 1400                            | 2800   | 4200    | 5600    | 7000    | 8400    |
| Dining Room<br>(dishwashers) | 600                             | 1200   | 1800    | 2400    | 3000    | 3600    |
| Laundry                      | 5500                            | 11,000 | 16,500  | 22,000  | 27,500  | 33,000  |
| Total                        | 34,150                          | 68,300 | 102,450 | 136,600 | 170,750 | 204,900 |

All values are in U. S. gallons per day

As can be seen in Table 1, the total flow for each 1000-man increment is 34,150 gpd, or approximately 35 gpd/capita. This value is on the lower end of the 35 to 100 gpd/ capita range given in the Statement of Work. The figure of 35 gpd/capita can be compared with the range of 20 to 90 gpd/ capita which is generally reported for domestic wastewater flows. It should be noted that such factors as a minimal water use toilet and the fact that water services are not as accessible serve to lower the per capita use figure. On the other hand, instances can be envisioned in which photographic processing will be a much larger operation than assumed for a typical bare base, thus raising the per capita figure above 35 gpd. In this event additional waste treatment modules can be provided with little difficulty to treat the excess flow.

## SECTION III

### WASTEWATER CHARACTERISTICS

Data have been compiled on some of the waste streams in the form of range of values. On other streams, it was necessary to estimate characteristics based on analysis of similar types of wastes.

Many of the data were supplied by the Environmental Health Laboratory, Kelly Air Force Base, Texas. Other sources of information were textbooks on industrial wastes and Military Environics (Ref. 2).

#### 1. PHOTOGRAPHIC WASTEWATER

The following data have been compiled for photographic wastewaters on the basis of the assumptions listed in Section II. It should be noted that the EA-4 chemical process is assumed to be the only process used and that the primary source of information has been Reference 8.

TABLE 2  
COMPOSITION OF PHOTOGRAPHIC WASTEWATER

| Process Waste Solution                    | BOD<br>mg/l | COD<br>mg/l | FLOW<br>l/min   | Contribution<br>to the total stream |            |
|---|-------------|-------------|-----------------|-------------------------------------|------------|
|   |             |             |                 | BOD <sub>5</sub> (mg/l)             | COD (mg/l) |
| Prehardener                               | 66,500      | 100,000     | .15             | 394                                 | 593        |
| Neutralizer                               | 8200        | 20,000      | .12             | 40                                  | 99         |
| First Developer                           | 9600        | 260,000     | .44             | 166                                 | 4,490      |
| First Stop Bath                           | 22,800      | 24,000      | .56             | 505                                 | 532        |
| Color Developer                           | 11,900      | 35,000      | .56             | 264                                 | 775        |
| Second Stop Bath                          | 13,200      | 29,000      | .50             | 260                                 | 572        |
| Bleach                                    | 900         | 41,000      | .15             | 5                                   | 242        |
| Fixing Bath                               | 35,000      | 136,000     | .15             | 207                                 | 805        |
| Washwater                                 | -           | -           | 22.71 (6.0 gpm) | -                                   | -          |
| Total Stream                              | 1841        | 8108        | 25.35 (6.7 gpm) | -                                   | -          |
| Concentrated Stream (excluding washwater) | 17,850      | 78,500      | 2.64 (0.7 gpm)  | -                                   | -          |

Since it may be desirable to treat the washwater separate from the concentrated wastewater, it is of interest to know the characteristics of each fraction. The anticipated BOD<sub>5</sub> and COD of the concentrated fraction are given in Table 2; these were calculated assuming that all components of the total wastewater flow would be concentrated in this fraction. It should be noted that this is an estimate based upon engineering judgment and contact with individuals from Eastman Kodak Company. Attempts to determine characteristics of the washwater fraction were not successful. Certain of the components of the various baths and developers are present in the washwater but the concentration of these will undoubtedly be small with respect to the concentrated fraction. An analysis of metals in the concentrated fraction is given in Table 3 (Ref. 8).

TABLE 3  
METALS IN PHOTOGRAPHIC WASTEWATER (CONCENTRATED FRACTION)

| Metal     | Concentration<br>mg/l | Metal    | Concentration<br>mg/l |
|-----------|-----------------------|----------|-----------------------|
| Lead      | 0.57                  | Cadmium  | 0.09                  |
| Copper    | 0.05                  | Silver   | 214.00                |
| Manganese | 0.05                  | Chromium | 2.66                  |
| Zinc      | 0.30                  | Iron     | 15.00                 |

## 2. AIRCRAFT AND VEHICLE WASHRACK WASTEWATERS

### a. Aircraft Washrack Wastewater

The constituents of aircraft washrack waste can be subdivided into two basic fractions. The first fraction is composed of those materials in

the dirt films which consist of oil, grease, soil, oxidized metal particles and salts. The second fraction consists of materials used to clean the aircraft surface and is composed primarily of detergents and solvents. Because of the action of the emulsifier found in the detergents, some oils and greases become stabilized emulsions which are difficult to remove from the wastewater.

The major water pollutants discharged from aircraft washracks find their origin in the cleaning and treating materials employed on the rack. Prior to about 1958, the U. S. Air Force used a commercially manufactured crude soap (often referred to as gunk) mixed with kerosene ( a refined petroleum distillate) for washing aircraft. To minimize oil pollution, most Air Force installations are now using improved alkaline water-base cleaner for washing aircraft. This cleaner is made up of compounds such as aromatic hydrocarbons, trisodium phosphate, pH buffers such as caustic potash, emulsifying agents and glycol derivatives. The emulsifying agents are non-ionic, cationic or anionic detergents, which are biologically degradable, and fatty acids combined with caustic potash to form a potassium soap.

Some useful data on the characteristics of aircraft and vehicle washrack wastewater have been obtained from a series of technical reports prepared by the USAF Environmental Health Laboratory at Kelly Air Force Base (Refs. 43, 44, 45, 46). The technical reports are the results of field surveys at many fixed Air Force installations. Considerable difference in operation and maintenance at each base produced significant variation in waste characteristics and quantity. Specifically, these differences include the degree and type of operation (e.g., partial or complete washing, paint stripping, engine parts cleaning, degreasing and over-hauling), inclusion or exclusion of storm water, variation in type

of cleaning agents and, most importantly, the occasional use of skimmers to remove free oil prior to sampling.

In view of the simple washing operation anticipated at a bare base, data from George, Dover and McChord Air Force bases were selected as being characteristic of those expected for a bare base. Table 4 summarizes these data. At George Air Force Base the samples were collected in a manner which excluded free oil and grease and, to some extent, settleable solids. The analyses were based on six daily composite samples. Average flow was 78,000 gpd of which 10,000 gpd was storm run off. Washing was done on four aircraft washracks. Detailed activities on the washrack were not specified; however, the base is a Tactical Air Command installation.

TABLE 4  
AIRCRAFT AND VEHICLE WASHRACK WASTEWATER

| Parameter       | Concentration* | Parameter    | Concentration |
|-----------------|----------------|--------------|---------------|
| COD             | 420-916        | Phenol total | .02-6.7       |
| BOD             | 135-300        | Cr           | .08-1.9       |
| TDS             | 500-580        | Cu           | .04-.14       |
| SS              | 27- 60         | Cd           | .02-.03       |
| Oil/grease      | 4- 50          | Ag           | - .09         |
| MBAS            | 3-160          | Fe           | .1-.7         |
| NO <sub>3</sub> | 1- 7           | Pb           | .1-.2         |
| PO <sub>4</sub> | 18-105         | Zn           | .1-.8         |
|                 |                | pH           | 7-8           |

\*All values in mg/l except pH

At Dover Air Force Base three planes were washed daily from which an average of 30,000 gallons of wastewater were produced. Washing was done both in the corrosion control facilities and outdoors on a large washrack. Samples were taken from the effluent of free oil separators. The values represent an average of 13 days of samples. During the period 17 planes were completely washed, 9 completely washed with corrosion removal, 7 partially washed and 1 paint stripped. Samples were diluted by snow during the last 2 days of sampling (flow doubled).

At McChord Air Force Base two planes were washed daily. The established flow was 8000 gpd, most of which results from the washing operation. Samples were taken from the effluent of the free oil separator.

b. Vehicle Washrack Wastewater

Examination of the reports (Refs. 43, 44, 45, 46) revealed wide variability in quantity and characteristics of such wastewater from installation to installation. However, the general characteristics of the vehicle washrack wastewater were very similar to those of the aircraft washrack wastewater, and it was assumed that they can be used interchangeably. Table 4 summarizes these data.

### 3. HOSPITAL WASTEWATER

Data on this waste stream were received from Garrett Airesearch Corporation, Phoenix, Arizona, who has developed a wastewater management system for the Mobility Army Surgical Hospital (MASH) (Ref. 47). The data reported in Table 5 are the average and the range of daily composite values for a five-source hospital. The wastewater sources included in the composite sample are 1) the shower and lavatory wastewaters, 2) the X-ray wastewaters, 3) the operating room and central materials supply

TABLE 5  
ANALYSIS OF FIVE-SOURCE HOSPITAL WASTE STREAM

| Analysis                            | Concentration <sup>*</sup><br>(Ave.) | Concentration <sup>*</sup><br>(Range) |
|-------------------------------------|--------------------------------------|---------------------------------------|
| Alkalinity                          | 104                                  | 0 - 268                               |
| Alkyl Benzene Sulfonate (ABS)       | 75                                   | 57 - 88                               |
| As                                  | 0.001                                | 0.001 - 0.002                         |
| Ba                                  | 0.12                                 | 0.02 - 0.31                           |
| Cd                                  | 0.02                                 | 0.01 - 0.04                           |
| Ca                                  | 15.0                                 | 10.0 - 25.0                           |
| Cr                                  | 1.05                                 | 0.80 - 1.60                           |
| Conductivity (umhos/cm)             | 1530                                 | 1100 - 2200                           |
| COD                                 | 870                                  | 550 - 1400                            |
| Cu                                  | 0.04                                 | 0.02 - 0.05                           |
| Greases and Oils                    | 43.4                                 | 36.8 - 53.0                           |
| Fe                                  | 0.33                                 | 0.22 - 0.47                           |
| Pb                                  | 0.32                                 | 0.22 - 0.52                           |
| Mg                                  | 15.0                                 | 15.0 - 25.0                           |
| Mn                                  | 0.03                                 | 0.008 - 0.14                          |
| pH (pH units)                       | 7.5                                  | 7.3 - 8.5                             |
| PO <sub>4</sub>                     | 166                                  | 106 - 315                             |
| K                                   | 34.6                                 | 29.4 - 36.6                           |
| Na                                  | 360                                  | 330 - 410                             |
| SO <sub>4</sub>                     | 28                                   | 1.2 - 103                             |
| Turbidity (JTU)                     | 42                                   | 25 - 80                               |
| Coliforms (10 <sup>5</sup> /100 ml) | 1.8                                  | 1.2 - 2.4                             |

\* mg/l except as noted

wastewaters, 4) the kitchen wastewaters and 5) the laboratory wastewaters. It is assumed for purposes of this report that the wastewater from a bare base hospital will be similar to those given in Table 5.

#### 4. HUMAN WASTEWATER

Two sources of information on this waste stream have been studied. Data were received from the Garrett Airesearch Corporation, Phoenix, Arizona, whose Wastewater Management System for the Mobile Army Surgical Hospital incorporates separate handling of the human wastes, and from Military Environics (Ref. 2). The data in both cases are limited to only the solids concentration, and even in this respect differ greatly. However, since it is anticipated that this waste stream will be incinerated, this inconsistency in the data will not be overly significant. Following is a tabulation of data:

| <u>Source of Information</u>                | <u>Solids Concentration<br/>(percent)</u> |
|---|---|
| Garrett Airesearch Corporation (Refs. 5, 6) | 15  |
| <u>Military Environics</u> (Ref. 2)         | 5.6 - 6.7                                 |

#### 5. LAUNDRY WASTEWATER

The Environmental Health Laboratory at Kelly Air Force Base, in reply to our requests for data on these wastewaters, advised us that such wastes are very similar in composition to commercial laundry wastewaters. Accordingly, the ranges of values in Table 6 have been compiled from two texts on industrial wastewaters (Refs. 9, 10).



TABLE 6

## CHARACTERISTICS OF COMMERCIAL LAUNDRY WASTEWATERS

| Analysis                | Concentrations<br>mg/l except as noted |
|-------------------------|--|
| pH (pH units)           | 9.0 - 10.3                             |
| Total Alkalinity        | - 511                                  |
| Alkalinity above pH 7.0 | 60 - 250                               |
| Total Solids            | 800 - 2100                             |
| Volatile Solids         | - 1500                                 |
| BOD, 5 day              | 370 - 635                              |
| Suspended Solids        | 210 - 540                              |
| Oil and Grease          | 170 - 550                              |

## 6. KITCHEN WASTEWATER

Data on this waste stream are reproduced in Table 7 as reported by Gouveia and Hooton (Ref. 11) who analyzed the kitchen waste stream at Williams Air Force Base, Arizona. It is not clear from the paper whether the waste stream contained ground garbage. However, it is the opinion of the investigators that it did not.

TABLE 7  
CHARACTERISTICS OF KITCHEN WASTEWATER

| Analysis                                    | Concentration<br>mg/l except as noted |
|---|---------------------------------------|
| Ca  | 41                                    |
| Mg  | 8                                     |
| Na  | 171                                   |
| K   | 10                                    |
| Fe (Dissolved)                              | 1.3                                   |
| NH <sub>3</sub>                             | 4                                     |
| SiO <sub>2</sub>                            | 50                                    |
| Cl  | 170                                   |
| SO <sub>4</sub>                             | 66                                    |
| HCO <sub>3</sub>                            | 181                                   |
| CO <sub>3</sub>                             | 0                                     |
| PO <sub>4</sub> , Total                     | 28                                    |
| PO <sub>4</sub> , Ortho                     | 25                                    |
| Oil and Grease                              | 878                                   |
| Dissolved Solids (Calculated)               | 730                                   |
| Dissolved Solids (Evaporation)              | 1071                                  |
| Suspended Solids                            | 2498                                  |
| Total Solids                                | 3552                                  |
| Chemical Oxygen Demand                      | 2800                                  |
| pH (pH units)                               | 6.0                                   |
| Specific Conductance (micromhos/cm at 25°C) | 950                                   |

## 7. COMPOSITE WASTEWATERS

It was not possible to obtain characteristics of the shower and lavatory wastewater and the dining room wastewater. Analysis of the means of treatment of the various waste sources, as reported in the following sections, resulted in the decision to treat several waste sources in the same treatment unit. To determine the characteristics of the composite waste which would be the influent of this treatment unit, several assumptions were necessary. The first of these was that the combined hospital, shower, lavatory, kitchen, dining room, laundry, and human wastes would approximate a typical domestic waste when corrected for the reduced volume of water used by the Air Force. The per capita contributions assumed for this calculation were (Ref. 23)

$$\text{BOD}_5 = 0.17 \text{ lbs/day (77 gms/day)}$$

$$\text{Suspended Solids (SS)} = 0.20 \text{ lbs/day (91 gms/day)}$$

Since these contributions include human wastes, it is necessary to subtract the quantities resulting from human wastes. It is assumed for this source that 1.6 liters of urine and 0.1 liters of feces are excreted per capita per day, and that this quantity of urine and feces combined has a  $\text{BOD}_5$  of 29 gms. Assume further that 2 gms of toilet paper are used per capita per day which have a  $\text{BOD}_5$  of 2 gms. Thus, the  $\text{BOD}_5$  contribution excluding human waste is

$$77-31 = 46 \text{ gms/capita-day of } \text{BOD}_5$$

Similarly, the contribution of human waste to the suspended solids concentration will likely be reduced to

$$\text{feces} \quad 35 \text{ gms/day}$$

$$\text{toilet paper} \quad \underline{2 \text{ gms/day}}$$

$$37 \text{ gms/day}$$

yielding suspended solids in the combined waste excluding human waste of

$$91-37 = 54 \text{ gms/capita-day}$$

For a 1000 man base, given the flows calculated in Section II of

|                     |            |
|---------------------|------------|
| hospital            | 500 gpd    |
| shower and lavatory | 20,000 gpd |
| kitchen             | 1400 gpd   |
| dining room         | 600 gpd    |
| laundry             | 5500 gpd   |

the  $BOD_5$  concentration of the composite flow is 434 mg/l and the suspended solids concentration is 510 mg/l. Since the waste flow has been assumed proportional to base size in all cases, these concentrations apply as well to base sizes of 1000 to 6000 men.

Study of the best means of treating aircraft and vehicle wash-rack wastes and photographic washwaters has shown that it may be desirable to treat these wastes with the combined stream. Because the photographic washwaters are likely to be very dilute and comparatively small in quantity, it was assumed that inclusion of this waste in the composite stream would cause no significant change in characteristics.

Adjustments in the  $BOD_5$  and SS concentrations for inclusion of aircraft and vehicle washrack wastes were made as follows for a 1000-man base:

|  |            |
|--|------------|
| Domestic type wastes (including shower, lavatory, kitchen, dining room, laundry) | 28,000 gpd |
| Vehicle and aircraft   | 4000 gpd   |

Average values of  $BOD_5 = 230$  mg/l and  $SS = 60$  mg/l are taken from Table 4 for the washrack wastes. On this basis, a composite  $BOD_5 = 410$  mg/l and  $SS = 454$  mg/l was calculated.

The increments of total solids per use cycle is particularly important if the wastewater is to be treated for reuse. Since incremental total solids data were not available for several of the waste sources, an estimate of 800 mg/l has been made. It should be noted that this figure should be determined accurately in future research.

A summary of the above calculations is given in Table 8 for the composite waste stream.

TABLE 8  
CHARACTERISTICS OF THE COMBINED WASTE STREAM

| Source  | BOD <sub>5</sub> (mg/l) | SS(mg/l) | Total Solids<br>(Increment added<br>per use) (mg/l) |
|---|-------------------------|----------|---|
| Hospital, shower, lavatory<br>kitchen, dining room, laundry                             | 434                     | 510      | 750   |
| All of above plus photographic<br>washwater and aircraft and<br>vehicle washrack wastes | 410                     | 454      | 800   |

## SECTION IV

### CHARACTERISTICS OF RESIDUES FROM WASTEWATER TREATMENT PROCESSES

The composition of residues varies widely depending on a number of factors. With given characteristics of wastewater, the types of sludges which are expected can be classified according to the treatment process in which they are formed.

#### 1. GRIT, SCREENINGS AND SKIMMINGS

Grit can be described as small inorganic solids that are removed from wastewater by gravity settling. It is comprised of sand, silt, gravel, ashes, coffee grounds and similar substances. Grit can be removed by a grit chamber at the head of the treatment plant, or by the use of hydroclones (Ref. 17) which effect their removal from solids settled in primary sedimentation basins. The moisture content of grit varies from 14 to 34 percent and the organic content ranges from 15 to 50 percent (Ref. 18). Burial with or without washing is the most common method of ultimate disposal.

Screenings consist mainly of rags, sticks and garbage. They are characterized by a moisture content of 85 to 95 percent and an organic content of 50 to 80 percent (Ref. 18). Where grinding of screenings is not practiced, a sanitary means of disposal is required because of the high organic content of these solids. Burial and incineration are alternative methods of disposal.

Skimmings are floating materials collected at the surface of sedimentation basins. They are characterized by high concentrations of grease and fibrous trash, a moisture content of 60 to 90 percent and

a volatile solids concentration of 90 to 95 percent (Ref. 18). They are usually disposed of by burial, digestion or incineration.

## 2. FLOTATION SLUDGE

The solids production in a flotation unit is largely dependent upon the waste characteristics, the type and amount of chemicals used and the solids removal efficiency of the unit.

Data on the performance of flotation units used for concentrating activated sludge (Ref. 19, 20) indicate that floated solids concentrations average about 4.6 percent at plants which do not condition with polymers. Plants which used polymers produced an average sludge concentration of 5.8 percent.

## 3. PRIMARY SEDIMENTATION SLUDGE

The fresh solids from primary settling basins contain most of the settleable solids in the raw wastewater. These constitute about 60 percent of the suspended solids in the wastewater. Their volatile content is about 70 percent and they can be removed at about 5 percent by weight. In general, the dewatering characteristics of these sludges are better than those of biological sludges.

## 4. CHEMICAL PRECIPITATION SLUDGE

This sludge results from application of coagulating chemicals to raw wastewater and includes precipitated chemicals and up to 95 percent of the suspended solids originally in the wastewater. By elevating pH through the use of chemicals such as lime, many metals present in the wastewater form insoluble hydroxides or carbonates. In addition to calcium and magnesium, strontium, barium, aluminum, trivalent chromium,

manganese, iron, cobalt, nickel and copper can be precipitated in this manner.

Sludges from chemical coagulation processes may be dilute. They may have a solids concentration as low as 1 percent, depending on the coagulant used (Ref. 21), or as high as 10 percent as in the precipitation of calcium carbonate in the lime softening process. The presence of magnesium precipitates appreciably reduces achievable solid concentrations. The precipitates from alum coagulation tend to be much more dilute than lime sludges. The volatile concentration of chemical precipitation sludge is highly dependent upon the amount of chemicals used and may be as low as 50 percent.

#### 5. WASTE ACTIVATED SLUDGE

Waste activated sludge presents the most troublesome feature in handling and disposal because it is very dilute and difficult to thicken. Solids concentrations of activated sludge range from 0.5 to 1.0 percent prior to thickening with a volatile content of about 70 percent.

#### 6. BRINES

Residual flows from tertiary treatment processes such as reverse osmosis, electrodialysis, ion exchange and distillation ordinarily contain 5000 to 15,000 mg/l of total dissolved solids (Ref. 22). The types of dissolved solids vary with the process and the characteristics of wastewater.



## 7. SUMMARY OF SLUDGE PROPERTIES

The production of sludge depends on the composition of wastewater, the type of treatment received and the efficiency of solids removal. Typical values of suspended solids concentrations of sludges for domestic wastewater are summarized in Table 9.

TABLE 9  
CHARACTERISTICS OF SLUDGE FROM DOMESTIC WASTEWATERS

| Type of Sludge                          | Solids Concentration<br>(percent by wt)* |
|---|--|
| Grit                                    | 66 - 86                                  |
| Screenings                              | 5 - 15                                   |
| Skimmings                               | 10 - 40                                  |
| Primary Sludge                          |  |
| Fresh                                   | 2.5- 5                                   |
| Gravity Thickened                       | 8 - 10                                   |
| Digested                                | 10 - 15                                  |
| Chemical Precipitation Sludge           |  |
| Fresh                                   | 1 - 10                                   |
| Activated Sludge                        |  |
| Fresh                                   | 0.5- 1.0                                 |
| Gravity Thickened                       | 2.5- 3.0                                 |
| Flotation Thickened                     | 4.6                                      |
| Mixture of Activated and Primary Sludge |  |
| Fresh                                   | 4 - 5                                    |
| Gravity Thickened                       | 5 - 10                                   |

\* Sources were Refs. 18, 19, 20, 21 and 23

Typical heat values of various sludges are listed in Table 10 (Ref. 24). Other pertinent properties such as physical, chemical and biological characteristics are described in later sections where the performance of each treatment process is evaluated.

TABLE 10  
HEAT VALUES OF SLUDGES

| Sludge                                | Combustibles<br>(Percent by wt) | Ash<br>(Percent by wt) | Heat Value<br>(Btu/lb Dry Solids) |
|---------------------------------------|---------------------------------|------------------------|-----------------------------------|
| Grease and Scum                       | 88.5                            | 11.5                   | 16,750                            |
| Fresh Sewage Sludge                   | 74.0                            | 26.0                   | 10,285                            |
| Fine Screenings                       | 86.4                            | 13.6                   | 8990                              |
| Ground Garbage                        | 84.8                            | 15.2                   | 8245                              |
| Rags                                  | 97.5                            | 2.5                    | 8050                              |
| Digested Sewage and<br>Garbage Solids | 49.6                            | 50.4                   | 8020                              |
| Digested Sludge                       | 59.6                            | 40.4                   | 5290                              |
| Grit                                  | 30.2                            | 69.8                   | 4000                              |

## SECTION V

### WASHRACK WASTEWATER TREATMENT

Typical characteristics of washrack wastewaters include emulsified oil and grease and significant amounts of free floating oil and grease. A high COD, intermediate BOD and low suspended solids are expected. Table 4 gives the anticipated characteristics subsequent to skimming and grit removal. In spite of the low suspended solids, emulsified oils and grease are particularly troublesome in treatment processes.

#### 1. REVIEW OF TREATMENT PROCEDURES

In existing military and civilian installations where a significant amount of such wastewater is generated, the methods of treatment depend largely on the characteristics of the wastewater, availability of municipal wastewater treatment plants, and the dilution provided by the receiving body. (Ref. 41). The methods of treatment have been diverse and can be broadly categorized as 1) physico-chemical treatment, 2) pretreatment with oil separators to remove free floating oil with subsequent discharge to receiving waters or 3) pretreatment followed by biological treatment. Biological treatment is used primarily when washrack wastes are discharged to municipal systems.

##### a. Physico-Chemical Treatment

The physico-chemical process for treating washrack wastewaters can be divided into three parts, 1) pretreatment by primary clarification, 2) chemical treatment, consisting of coagulation and dissolved air flotation and 3) effluent polishing. The primary clarifier in the pretreatment scheme is designed for a dual purpose in that it serves as an

oil separator for removal of free-floating oil and grease and as a settling unit for the removal of grit and readily settleable solids. This pretreatment unit is important in that it reduces chemical requirements, protects mechanical equipment and avoids scum and grit problems in subsequent treatment units (Ref. 42). With an overflow rate of approximately 700 gpd/ft<sup>2</sup> and properly designed skimming devices, the unit can remove essentially all settleable solids and free oil and grease.

The suspended solids, which are primarily stabilized emulsions, are removed after destabilization. Breaking an emulsion is analogous to the destabilization-aggregation phenomena which occur in a typical colloidal system in water and wastewater treatment. The repulsive forces due to electrical charges on the oil droplets are overcome by the coagulant and aggregation of the oil droplets and coagulant precipitates takes place either by mechanical entrapment or by other means (Ref. 48). A number of coagulants including aluminum salts, ferrous or ferric salts, lime, clay and a number of polyelectrolytes have been tried by many investigators. As anticipated, results on the efficiency of such chemicals were diverse. Couter et al. (Ref. 49) noted that ferric sulfate was the most effective chemical for breaking emulsified oils at U. S. Air Force installations. In certain cases conjunctive use of alum and lime was very effective (Ref. 42). A coagulant dosage of 80 to 150 mg/l was required (Refs. 42, 49). Following coagulant addition the flow is gently stirred in the flocculation basin to promote coalescence of the oil droplets.

Suspended chemical floc with entrapped or adsorbed oil particles or other solids must be removed by dissolved air flotation or sedimentation. Owing to the light nature of these particles, flotation has found increasing use in recent years (Refs. 42, 44, 48, 50). The surface loading rate

for the dissolved air flotation process ranges from three to ten times greater than that required for sedimentation.

Successful emulsion destabilization and subsequent phase separation reduces BOD, COD, and possibly heavy metals and phosphate concentrations. The BOD and COD of the effluent from the flotation process is likely to be high, however. Soluble organics are high initially and it is not known how many are removed by flotation. Some of the surface active compounds will undoubtedly be removed but many dissolved components will remain in the water.

Adsorption using activated carbon or synthetic resins is a likely process for removing the dissolved organic compounds although reports of its use for this purpose have not been found. Porous adsorbents are particularly susceptible to fouling by oil so it may be necessary to precede the adsorber with a granular filter to remove any droplets which escape the flotation unit. Effluent quality from the adsorber is estimated as COD of 10 to 50 mg/l, BOD of 10 to 50 mg/l and negligible suspended solids.

The sludge resulting from treatment of washrack wastewater consists of the free oil and grease skimmed from the surface of the primary clarifier, the sludge collected at the bottom of the primary clarifier and the sludge removed from the surface of the flotation unit. The normal practice of disposing of the wastes has been lagooning (Ref. 41). Use of a holding tank (Ref. 50) and acidification (Ref. 51) to further concentrate the combined residues has also been reported. Since the sludge from the flotation unit consists of chemical floc together with oil and particulates, acidification will dissolve most of the metal hydroxide, thereby concentrating the oil. However, it may also redissolve other unwanted

precipitates such as heavy metals and the liquor thus produced may be difficult to dispose of. Oil and grease is readily combustible so the waste sludge should be suitable for incineration.

b. Biological Treatment

In certain instances washrack wastewaters have been treated biologically in conjunction with municipal wastes. Although joint treatment with municipal wastes is not considered a possibility for the Bare Base Program, results of joint treatment studies are of interest because of implications with regard to biological treatability. The major problem in combining washrack wastewater with municipal wastewater for treatment has been the impact of oil substances, certain heavy metals and phenolic substances on biological waste treatment. Oil and grease may coat biological mass sufficiently to interfere with oxygen transfer from the liquid to the living cells (Ref. 52). Therefore, the degree of pretreatment that the wastewater should receive prior to municipal treatment depends on the amount of such substances and the dilution provided by the municipal wastewater. In any case, free floating oily substances should be removed prior to combining with municipal wastewater to prevent scum problems in the treatment units.

Emulsified oil and grease are destabilized by biological attack on the emulsifying agents. The finely divided particles then coalesce into larger particles which can be separated from the water (Ref. 52). In activated sludge processes, the coalesced oil and grease may float to the water surface and be carried out in the effluent of final settling tanks. An extensive study on the biological treatability of washrack wastewater conducted by the Regional Environmental Health Laboratory, Kelly Air Force Base showed the potential of treating such wastewater in typical domestic

treatment plants at dilution ratios of less than 1 to 1 (Refs. 45, 46, 53, 54, 55, 56, 57).

Biological treatment could also be relied upon to reduce the concentrations of dissolved organics.

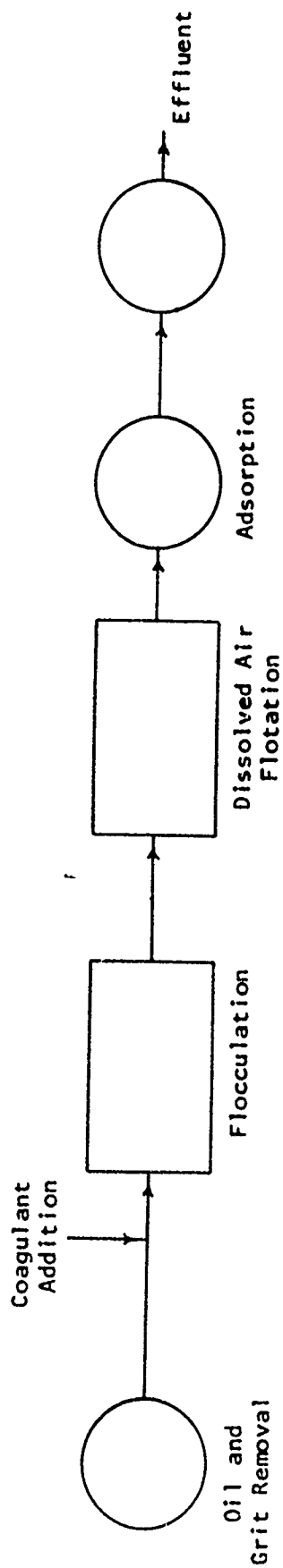
## 2. SELECTION OF THE RECOMMENDED TREATMENT PROCEDURE

Based on the preceding review of potential processes, three alternatives for treatment have been developed and are presented in Fig. 2. The physico-chemical alternative, which includes pretreatment for removal of grit and free-floating oil and grease, chemical clarification with flotation for solids removal, filtration and adsorption, is capable of producing an effluent suitable for reuse in washing vehicles and aircraft. The necessity of the filtration step should be evaluated in further research if this treatment scheme is to be used. The adsorption step would utilize either activated carbon or synthetic resins as the adsorbent; these adsorbents should be evaluated in further study if the physico-chemical scheme is used.

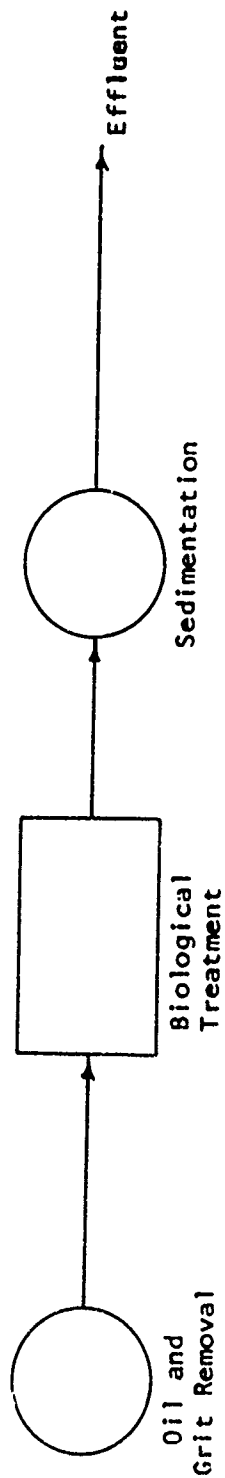
The cost of equipment for the physico-chemical alternative can be approximated using cost data presented by Crits (Ref. 108) and others. The volume of waste is 4000 gal/1000-man increment.

|   | <u>Capital Cost</u> |
|---|---------------------|
| 1) 1000 gal pretreatment tank                           | \$1200              |
| 2) chemical feeders, rapid mix, flocculation, flotation | \$2000 + flotation* |
| 3) filter   | \$ 500              |
| 4) adsorber   | \$ 500              |
| Plus 20% for adding equipment together                  | <u>\$ 850</u>       |
| TOTAL   | \$5040 + flotation  |

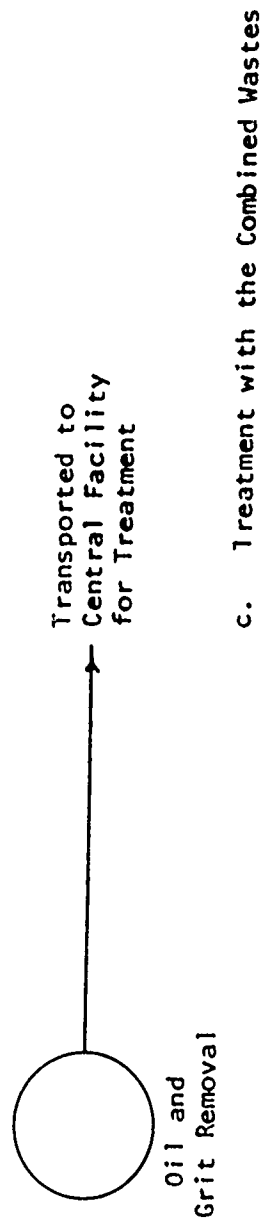
\* A flotation unit of this size is not made commercially; thus, a cost estimate is not available.



a. Physico-Chemical Treatment



b. Biological Treatment



c. Treatment with the Combined Wastes

Figure 2. Alternatives for Washrack Wastewater Treatment



Assuming that a loading of 1 lb COD/ 2 lb carbon is possible and that 500 mg/l COD is removed by the carbon, approximately 1000 lbs/mo of carbon will be required. If synthetic resins can be used as an adsorbent on-site regeneration may be possible. A small volume of regenerant, such as caustic or methanol, would then be required in place of the make-up carbon. Assuming a coagulant dose of 150 mg/l of a ferric salt, approximately 150 lbs/mo of coagulant would be required.

Separate treatment by the physico-chemical system is not recommended for several reasons. Poor treatment efficiency would probably be experienced because of the need for intermittent operation. Although some equalization can be achieved in the primary clarifier, 24 hour operation of the plant would not be likely. During periods of startup it is anticipated that effluent quality would be highly variable and unsatisfactory. Since the vehicle washrack is located some distance from the aircraft washrack, a system for transporting the waste would be required which would increase the complexity of the system. The system would also probably require the services of a person with a good knowledge of the processes to start operation and to adjust parameters to achieve good efficiency. Since the system developed in Section VIII for treatment of the combined wastes involves essentially the same processes, it appears desirable to transport the waste to a central facility for treatment in accordance with alternative c in Fig. 2 and as is developed in more detail below.

Alternative b, which involves pretreatment for removal of grit and free floating oil and grease, biological treatment and sedimentation followed by either disposal to the environment or by effluent polishing by filtration and adsorption, is not developed in any detail because it is not

suited for the intermittent nature of the flows. Biological treatment can not be started and stopped on a daily basis. If the wastewater flow were continuous, this system would warrant more consideration, however.

The recommended system involves pretreatment and transportation to the central treatment unit for treatment with the combined waste. The wastes should be collected in a holding tank with approximately one day's capacity. This tank should be equipped with the necessary skimming device to remove the free floating oil and grease. Grit removal can be accomplished manually when the tank is drained. The wastewater would then be transported by truck to the central treatment unit on a daily basis. This procedure is most consistent with the requirement that the system be easy to operate, easily set up and dismantled and of low cost. The waste is also compatible with the combined wastes and thus should not affect treatment efficiency in the central unit.

In transporting the partially treated wastewaters to the central waste treatment facility, either trucking or pipeline could be used. The distance of transport is slightly more than one mile. In the event that a pipeline were to be used, the cost of pumps, pipe and pipe insulation would be approximately \$7500. The number of manhours required to place, or redeploy, over one mile of pipe is high, particularly if the pipe is to be buried, and is inconsistent with bare base criteria. If the piping is placed on the surface, there is a danger of damage by trucks, etc. For these reasons, trucking of the waste should be used.

Holding tanks of 1000 gal capacity for the vehicle wastewater and 3000 gal for the aircraft washwater are required for each 1000-man increment. The expected cost for holding tanks is approximately \$1600 per 1000-man increment. A truck would be required to make one transfer per day as well.

It is estimated that 30 gal/1000-man increment of oil and grit would be removed from the holding tanks each day. The skimmings would be collected in drums and transported to the sludge incinerator for burning while the grit could be buried in a land fill as needed.

The recommended system is considered most consistent with bare base criteria in that it requires minimal on-site construction, is readily redeployable, is operable in extremes of climate and geographical conditions and will have the lowest overall cost because of the economy of scale obtained by treatment with the combined wastes.

## SECTION VI

### PHOTOGRAPHIC WASTEWATER TREATMENT

As indicated in Section III, photographic wastewaters were particularly difficult to characterize. It was determined, however, that the total waste stream can be separated into a concentrated stream, constituting approximately 10 percent of the total flow, and a washwater stream. The characteristics given in Table 2 for the combined stream include a  $BOD_5$  of 1841 mg/l and a COD of 8108 mg/l. The  $BOD_5$  for the concentrated stream is 17,850 mg/l and the COD is 78,500 mg/l. A 6000-man base produces approximately 8600 gpd of washwater and 1000 gpd of concentrated waste.

#### 1. WASTE REDUCTION

Reduction of volume and strength of the waste requiring treatment is of much importance. This should be possible, particularly if a more extensive investigation into promising areas is carried out. The bleach, for example, contains a large concentration of ferricyanide complex. In the spent bleach much of the ferricyanide has been reduced to ferrocyanide. In accordance with usual procedures, the bleach is disposed of when the ferrocyanide concentrations become too large. Several processes can be used to reconvert ferrocyanide to ferricyanide. Oxidizing agents such as persulfate and ozone have potential for this conversion (Ref. 65) but it remains to be determined whether these substances can be used reliably.

Some developing solutions can be regenerated by ion exchange procedures to remove bromides which accumulate and destroy their effectiveness (Ref. 65). Again, the reliability of this process apparently needs to be demonstrated.

Silver recovery for reuse is frequently carried out at photographic processing installations. Silver accumulates in bleaches and fixers from which it can be removed by precipitation, electrolysis or ion exchange (Ref. 65).

The potential for reduction of volume of washwater also requires close study. Also, the use of squeegees to reduce carryover of processing solutions from tank to tank seems reasonable (Ref. 65). The potential for reuse of washwaters also should be investigated.

## 2. WASTE TREATMENT ALTERNATIVES

There is little information available on the efficiency of various processes in treating photowastes. It is felt that much more is known than is available in the literature, but that much of this information is proprietary in nature. Accordingly, the alternatives are discussed from the standpoint of what should be possible rather than reported studies.

A treatment procedure which is untested as far as the authors of this report can ascertain consists of treating the combined waste stream with an oxidizing agent. This would probably involve treatment with a chemical such as ozone to convert many of the organic species to  $\text{CO}_2$ , the cyanide ion to  $\text{CO}_2$  and  $\text{N}_2$  and substances such as hydroxylamine and sulfite to inert compounds. Once the obnoxious components have been destroyed, it should be possible to discharge this waste directly to the environment. Sufficient dilution capacity must be available in streams to dilute the inorganic solids to concentration levels near those in the receiving water. The total inorganic solids concentration in the treated waste stream would probably be near 4500 mg/l (personal communication from Kodak Co.). Situations may arise where discharge of waters with these levels of salts

may not be permitted, however, and alternate treatment procedures should be available in this event. If a bare base were located near brackish or salt water, disposal of the treated effluent to these waters would be possible.

Another possible treatment method involves separation of the total stream into two fractions: the washwater and the concentrated wastes. The concentrated stream would have a  $BOD_5$  of approximately 17,850 mg/l, a COD of 78,500 and a total inorganic solids concentration of 35,000 to 45,000 mg/l. In the absence of other information, the washwater was assumed to have characteristics similar to that of the combined wastes, i.e.  $BOD_5$  of 400 mg/l, COD of 800 mg/l, suspended solids of 450 mg/l and total inorganic solids of 800 to 1000 mg/l. The concentrated fraction can be treated by incineration with the sludge from the sewage treatment process. The washwater stream can be treated with the laundry, shower, lavatory, hospital, washrack, and kitchen wastes. The feasibility of this approach requires testing, particularly because of the lack of the washwater characteristics and also because of the lack of actual operating data on such a process. Potential problems involved in incineration of the concentrated wastes include particulate and heavy metal emissions and ultimate disposal of the ash (Ref. 109).

### 3. SELECTION OF THE RECOMMENDED TREATMENT PROCEDURE

The recommended treatment procedure involves separation of the total waste stream into the concentrated stream and the washwater stream. The concentrated stream will be collected in a holding tank (165 gal capacity is sufficient for a day's production at a 1000-man bare base) and transported by truck to an incinerator.

The washwater will be collected in a holding tank and transported by truck to the waste treatment unit for the combined wastes. Analyses similar to those for washrack wastes show trucking to be more economically feasible than piping and more in keeping with the ease of set-up and redeployment criteria. Approximately 1500 gal of tank capacity would be required for each 1000-man increment.

The costs associated with this procedure are as follows for each 1000-man increment in size:

Fuel (1 gal fuel/4 gal waste), 43 gpd, \$4.30/day

Holding tank, 1500 gal, \$300

Holding tank, 165 gal, \$200

Plus trucking expense

This procedure will have the added benefit of minimizing frozen lines which could result in the arctic. The trucking of wastewater is more in keeping with the criteria in that the system can be easily assembled and disassembled and will be easy to operate.

Problem areas involving emissions of particulates and metal oxides during incineration require further consideration. Pollution control devices are available to eliminate particulates, but it may be necessary to remove heavy metals such as silver prior to incineration to prevent their volatilization. Any measures which could be taken to reduce the quantity and strength of the wastes being incinerated would, of course, be useful in controlling emissions. It is also probable that fluidized bed incinerators could be used to minimize emissions (Ref. 109).

## SECTION VII

### HUMAN WASTE TREATMENT

#### 1. REVIEW OF TREATMENT PROCEDURES

Treatment of human wastes can be accomplished in conjunction with other wastes, such as in a typical municipal system, or separately by processes such as incineration, wet-air oxidation, digestion, etc. The type and economy of treatment depend to a large degree on the amount of water used to carry the waste as well as other factors. The processes which have potential application in the Bare Base Program are discussed in the following text and are shown schematically in Fig. 3.

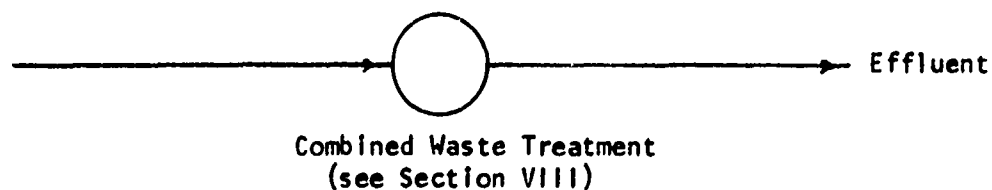
##### a. Separate Treatment vs. Treatment with the Combined Wastes

Human wastes can be treated with the combined wastes by biological or physico-chemical processes which are discussed in Section VIII. Human wastes are readily biodegradable and are amenable to physico-chemical treatment. In the event that human wastes were to be treated with the combined wastes, the BOD load at the central treatment facility would be increased by approximately 40 percent. The increase in volume of flow would depend significantly on the means of waste carriage.

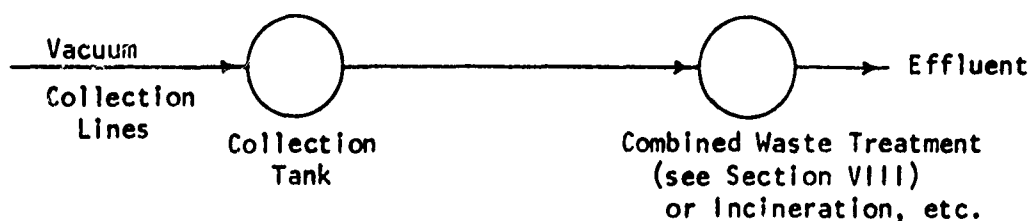
There would be significant economic advantages in treatment with the combined wastes. Elimination of separate treatment facilities would permit a significant economy of scale. If the USAF decides to use this alternative, the same treatment procedures discussed in Section VIII could be used. Adjustments would of course be necessary for increased BOD load.

In spite of the economic advantage, it is considered desirable to treat human wastes separately from the other wastes at a bare base for several reasons. The foremost of these is that decentralized treatment is

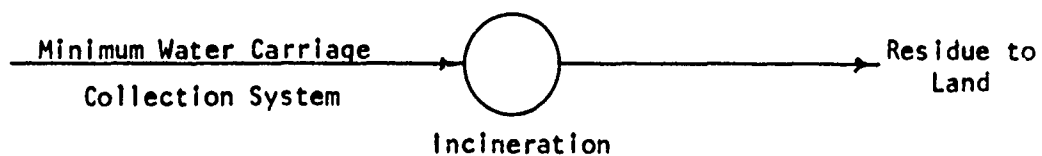




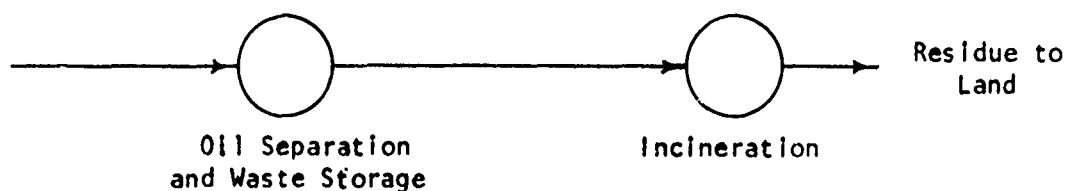
a. Treatment with the Combined Wastes



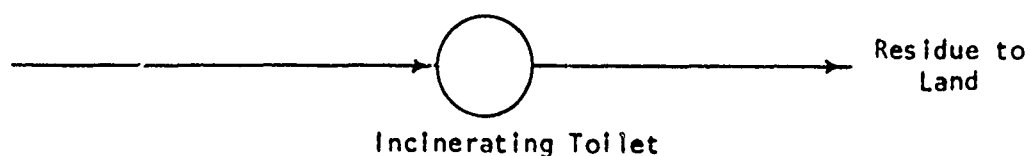
b. Vacuum-Collection System



c. Minimum Water Carriage Collection System



d. Oil Carriage Collection System



e. Incinerating Toilet System

Figure 3. Alternatives for Human Waste Collection and Treatment

made possible for the waste source of greatest potential health hazard. The human waste portion of the total treatment system could thus be made relatively invulnerable to complete destruction by bombing and this would make it possible to dispose of human wastes under the most adverse conditions. Additionally, if the wastewater is to be reclaimed for reuse, removing the human waste from the rest of the wastewaters would remove some of the health risk. It would virtually eliminate the threat imposed by pathogenic bacteria and virus which might not be removed due to failure of the water reclamation system. The effluent from the reclamation unit would be psychologically more acceptable for reuse as well.

Other advantages resulting from separate treatment of the human waste include the fact that a significant reduction (approximately 40 percent) in the BOD loading of the waste treatment facility for the central waste treatment unit occurs. This reduction in strength should make it possible to achieve a higher quality effluent. The central waste treatment unit will also be more acceptable aesthetically because the odorous human waste has been eliminated.

#### b. Vacuum Collection System

In an effort to reduce the amount of water associated with human waste in waterborn collection systems, the vacuum collection system has been developed. The system was developed in Sweden in 1959 and is currently being used for communities as large as 40,000 population (Ref. 111). National Homes Corporation has a license to market one such system, the SANIVAC system, in the United States. The system consists of a toilet bowl connected to a plastic pipeline and storage tank. The tank and pipeline have a pressure of 1/2 atmosphere: thus, when the valve is opened in the flushing operation the pressure on one side of the waste in the toilet

bowl is 1 atmosphere and on the other side is  $1/2$  atmosphere. Accordingly  $1/2$  atmosphere of pressure drives the waste through the collection line to the storage tank. Approximately  $1/3$  gallon of water is needed per flush as compared with 5 gallons in a typical household system. Because the waste is drawn through the pipe by a pressure differential, it is possible to install the collection line without major concern for maintenance of slope as is necessary in a gravity flow system. It should be possible to counter differences in elevation of approximately 10 ft with this system.

After collection of the waste in the storage tank, it is possible to transport the waste to a treatment facility by pipeline or truck, or to treat the waste adjacent to the storage tank. Biological treatment, physico-chemical treatment or incineration would all be possible for the waste from such a system.

#### c. Minimum Water-Carriage Collection System

The quantity of water for carriage of human wastes has been significantly reduced by the vacuum system, but it is reasonable to examine whether or not further reductions can be brought about. A recent article (Ref. 110) discusses the system which is commonly used in Japan. In this system, wastes from the toilet bowl drop directly into a storage tank situated below the housing facility. Presumably a small amount of water is required to flush the toilet bowl but this would probably be less than  $1/3$  gal per flush used in the vacuum system. Every three to four weeks a vacuum truck pumps out the storage tank and transports the waste to a central storage tank and treatment facility. Both the truck and the central treatment facility have odor control devices; the storage tank in the dwelling itself is well vented. It is claimed that the operation is essentially odor-free.

Various treatment methods can be used at the central facility once the waste has been delivered there by vacuum truck. Processes such as anaerobic digestion, wet air oxidation and incineration can reasonably be used. For bare base application, anaerobic digestion should not be seriously considered because it is a biological process which requires a very extensive start-up time and produces a fairly potent supernatant liquor which requires further treatment. Wet air oxidation similarly produces a fairly potent supernatant which requires further treatment. This further treatment can be biological or physico-chemical, but either of these will increase the complexity of the entire system. Incineration, holds much potential for bare base application in that it is readily started up, and the end product is an inert residue which can readily be disposed of in a sanitary landfill. Incinerators can also be readily transported. Incineration in this case is more feasible than in the case of the vacuum system because of the much smaller quantity of water used in the flushing process.

The Airesearch Division of Garrett Corporation has recently developed a system for the MUST hospital of the U. S. Army which includes minimum water carriage of wastes and batch incineration (Refs. 5, 6). This system is used in conjunction with a latrine facility. The latrine consists of six toilets located directly over two holding tanks. The toilet units are of the recycling flush-water type. Each tank is initially primed with water and bacteriostat for control of biological growth. A mixture of water and liquid human waste is recirculated for flushing purposes by means of a pump which is capable of separating the gross solids from the fluid. When the tank fills its contents are

manually discharged to the incinerator. Incineration is accomplished in two phases. In the first phase evaporation of the liquid takes place while in the second phase actual burning of the solids occurs. The combustible gas given off during the first phase aids in burning the solids. Auxiliary fuel is provided during the process until the exhaust temperature reaches 1600°F at which point the fuel supply is shut off; when the temperature drops to 1100°F auxiliary fuel is supplied again to maintain the desired temperature.

The physical dimensions of the incinerator are 3.5 ft x 3.5 ft x 3.0 ft with a 9.2-ft demountable stack. The burn cycle is automatic and includes shutdown and purge; operation and maintenance requirements are said to be minimal (Ref. 64). Weekly ash removal is required. The performance claimed for the unit includes a burning rate of approximately 5 gph with an auxiliary fuel requirement of 1.1 gph. The electrical power requirement is 350 watts (Ref. 64). The incinerator goes from ambient to 1300°F within 2 minutes after batch loading. The latrine facility as described before has two holding tanks, each with a 43-gallon capacity. Twelve gallons of flush water are required per use cycle. Research on the capacity of the incinerator is being carried out at the present time but a capacity of one incinerator/250 men appears reasonable.

#### d. Oil Carriage Collection System

Investigation of oil carriage of human waste followed by separation of the oil from the waste for reuse has been investigated in past years (Refs. 60, 62). The flushing fluid in this case must provide reliable transport of human waste and must be easily separated from the suspended feces and urine. Other desirable characteristics include

immiscibility of the fluid with water and that it be noncorrosive, non-toxic, nonflammable and nonvolatile. It should also have a low freezing point and not be objectionable in odor and color.

A recent development by Chrysler Corporation's Space Division includes an oil carriage collection system, a separation and storage tank and an incinerator for combustion of the separated waste (Ref. 63). Human wastes are flushed and transported by a water-immiscible oil to the separation tank where they are separated from the flushing medium. The undiluted waste is then combusted in the incinerator with the aid of auxiliary fuel.

The incinerator is essentially a batch-type, two-stage burning unit. The first stage involves heating the waste to evaporate the liquid and volatilize the solids. The temperatures maintained in the first stage range from ambient at the start of operation to 1400°F at the end. The vapors driven off from the first stage are continuously transferred to the 1400°F second stage for oxidation. Additional air is mixed with the vapors for complete oxidation. The residual in the first stage ignites in the presence of air at temperatures near 1000°F and is oxidized until only an inert residue remains. After burning the automatic temperature controller shuts down the burners and the next batch is added automatically, if necessary. Ash is manually removed from the boiler.

One of the most important features of the oil carriage system is its capability for delivering undiluted human waste to the incinerators. It minimizes the heat requirement for burning of the waste and keeps the physical size of the unit small.

Based on a prototype unit being constructed for the USAF for test purposes, the waste produced by 250 men can be incinerated in a unit

approximately 94 cubic feet in volume. The dimensions of this unit are 6 ft x 3.5 ft x 4.5 ft, including the incinerator and separation tank. The weight of the unit is less than 3000 lbs (Ref. 63). The design capacity of this unit is 100 gallons of undiluted human waste per day with 24-hour operation.

The fuel requirement for burning 4 gal of waste is 1.0 to 1.8 gal of diesel fuel. The time required for a complete batch cycle in this incinerator ranges from 40 to 46 min. Ash is removed manually once or twice a week. No harmful effect of unremoved ash on the burning of incoming waste has been noted. Ash production amounts to approximately 20 grams per gallon of waste.

The Chrysler unit is highly compact and appears to be very simple in operation. It does not have moving parts and seems to require minimum labor for operation and maintenance. However, at the expense of simplicity and compactness, the unit seems to have the disadvantage of inefficient heat utilization as do other batch-type incinerators. The theoretical auxiliary heat requirement for complete vaporization and oxidation of undiluted human waste while keeping the stack gas temperature at 1400°F is approximately 1240 BTU/gal. The actual heat requirement based on the above data ranges from 150 to 270 percent of the theoretical requirement. The probable reason for this heat loss seems to be retarded heat transfer into the mass of waste, heat loss through the stack gases, the requirement for excess air to promote complete oxidation, heat loss through refractories and a residual heat loss at the end of the batch operation.

Possible difficulties with the oil carriage system could stem from inefficient separation of the human waste from the oil and malfunctioning of the device used to make the separation. If the waste-oil separation

was inefficient, makeup oil would undoubtedly be required.

e. Incinerating Toilet Systems

Incinerating toilets have been widely studied for military applications (Ref. 2). In most cases the human waste is incinerated in the toilet bowl. According to an evaluation of commercially available incinerator toilets by the U. S. Naval Civil Engineering Laboratory (Ref. 59), deficiencies existed in terms of fuel consumption, odor destruction and reliability. Design criteria were established (Ref. 60) for better performance and include 1) a primary chamber with sufficient heat to accomplish evaporation, burning of the dried organic matter and heating of vapors to 1000°F and 2) a secondary chamber where vapors are subjected to temperatures above 1600°F with sufficient oxygen, turbulence and time for their destruction. Provided an efficient process to meet the above criteria can be devised, the major drawbacks of incinerating toilets include the need for protection of users during the combustion cycle, heat losses caused by intermittent operation and the necessity of flushless collection and conveyance (Ref. 58).

A modification of the incinerating toilet involves the use of fuel oil as flushing fluid (Ref. 61). The human waste-fuel oil mixture is finely ground prior to spraying into a burner. This process provides a simple method of waste collection and disposal but odors caused by fuel oil in the toilet room, fire hazard and clogging of burner nozzles were found to be major problems.



## 2. SELECTION OF THE RECOMMENDED TREATMENT PROCEDURE

The treatment system recommended for human waste consists of a minimum water carriage collection system and batch incineration at each latrine facility. Such a system would be operable at every conceivable bare base site in all extremes of geography and climate. It also should give a great deal of decentralization to the most critical part of the waste treatment system. The advantage of this system over the oil carriage system is that the waste-carriage medium separation step is not necessary, thus improving simplicity of operation. More fuel will be required than for the oil carriage system, however. Set-up time should be very minor and it should be easily operable in much less than 72 hours. Manpower will be required to open valves to transfer the contents of the collection tank into the incinerator; it should be possible for one man to handle this for base sizes up to 6000 men.

Because of the large number of incinerators that are required, capital cost for this system will be very high. A very tentative estimate of \$10,000 to \$30,000 for a 250 man-per-day unit was made by the Airesearch Division of Garrett Corporation. Seven of these units would be required for the first 1000 men in order to have an incinerator located with each latrine according to the base layout given in Figure 1. The corresponding capital cost is \$70,000 to \$210,000. For each additional 1000 men, however, only 4 units would be required and thus the cost on a per man basis would decrease with increasing base size. Fuel costs would also be high; assuming one gallon of fuel per 4 gallons of waste at a cost of 10 cents per gallon of fuel, a cost of 1.6 cents per capita per day is calculated.

The recommendation for this system should be qualified because of ongoing tests on both the minimum water collection system produced by Garrett Airesearch and the oil carriage system manufactured by Chrysler Corporation. The USAF is testing each system at the present time for application in the Bare Base Program. These evaluations should be used to confirm or reject the recommendation given here.

An alternative to the recommended system is the oil carriage system such as that manufactured by Chrysler Corporation. This system, too, is in its preliminary stages of development. Based on past experience some difficulty in separation of wastes from the oil is anticipated. Essentially the same number of these units would be required as for the Garrett Airesearch system. Slightly less fuel would be required in the case of this system because flushing water is not present.

It may be possible to develop lower cost systems if some decentralization of the system is sacrificed. Enlarged collection tanks could be used at each latrine and a vacuum truck could be used to collect wastes at 2 to 4-day intervals, similar to the Japanese system, and to transport the wastes to a central incinerator. Alternatively, the vacuum collection system could be used to collect the wastes and transport them to a storage tank next to a centralized incinerator. The vacuum would eliminate the trucking but would have the disadvantage of adding a collection system which would require extra installation time and maintenance. The vacuum system may also be limited by topography and it would also be more difficult to dismantle and redeploy. In the event that an incinerator of 4000 gpd burning capacity were used [such as the large incinerator made by

Garrett Airesearch (see Section X and Ref. 64)], one large incinerator could replace 3 to 4 of the small ones. A firm estimate of the cost savings can not be made until such time as good estimates of incinerator costs are available, however.

## SECTION VIII

### TREATMENT OF THE COMBINED WASTEWATERS

The combined wastes include lavatory, shower, kitchen, dining hall, hospital, photographic washwaters and vehicle and aircraft washrack wastewaters. They do not include human wastes or the concentrated fraction of the photographic process wastes; these latter wastes will undergo separate treatment. Either biological or physico-chemical processes can be used to remove the suspended and dissolved solids to the required degree as shown in Fig. 4. Microorganisms are used to accomplish the removal in biological treatment whereas chemical reagents, adsorbents and mechanical separation processes are used in physico-chemical treatment.

It should be noted that essentially the same pretreatment and disinfection procedures are required for both biological and physico-chemical treatment.

#### 1. PRETREATMENT

##### a. Grit Removal

Grit removal is a typical pretreatment process but it is not considered necessary in treatment of the combined wastes. Grit will be removed from vehicle and aircraft washrack wastes prior to its transport to the central treatment facility. The quantity of grit in the other incoming wastes will probably be small. There is a possibility of increased wear on equipment such as pumps because of the presence of this grit but this increased wear should be more than offset by the increased simplicity of operation owing to elimination of the grit removal process.

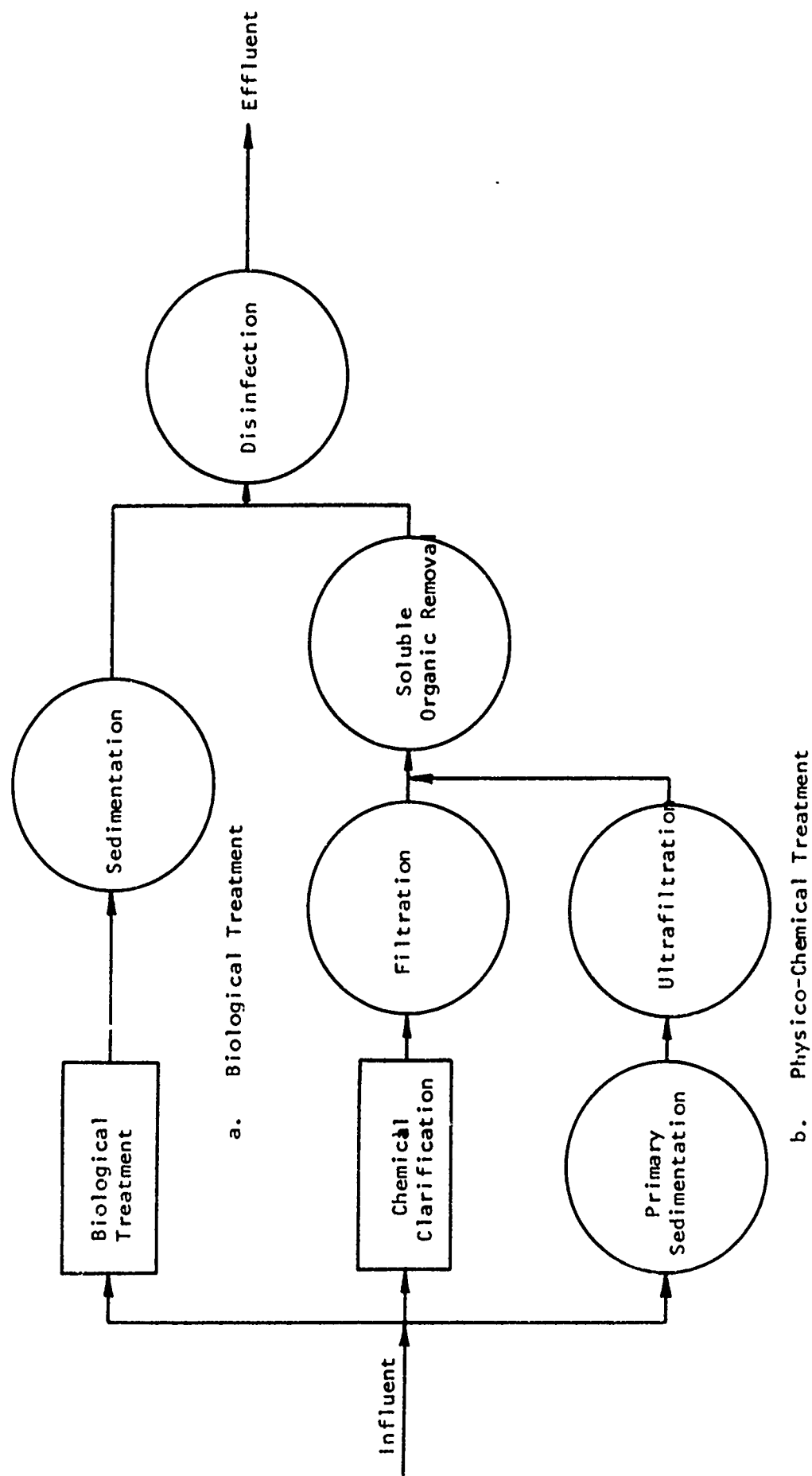


Figure 4. Alternatives for Combined Wastewater Treatment

In the event that grit removal is desired, an in-line degritter should be used. This device is compact and easy to operate. Its capital cost is approximately \$1000/1000-man increment.

b. Oil and Grease Removal

No process for oil and grease removal is necessary at the central treatment facility. Vehicle and aircraft wastes have been skimmed prior to transport to the facility. Kitchen wastes have the potential for oil and grease content so these wastes should be collected such that free oil and grease removal is possible. It may be necessary to pass these kitchen wastes through a grease trap.

c. Size Reduction

Size reduction is necessary to make the waste more homogeneous and easier to treat. Size reduction can best be accomplished by an in-line grinder at an estimated capital cost of \$500/1000-man increment.

d. Equalization

The purpose of equalization is to stabilize the chemical and physical characteristics of a waste prior to treatment. This is especially necessary if waste characteristics vary widely with time. Equalization can be accomplished by collection and mixing the waste in a large tank prior to treatment. This function is particularly important at installations such as a base because many wastes of different characteristics require treatment. Proper equalization permits a more uniform addition of chemical for removal of particulates and makes it possible to obtain a better quality of treated water. Equalization also adds flexibility to the treatment system in that additional chemical pretreatment, such as pH adjustment, can be readily accomplished.

Detention time is very important and depends on the nature and extent of variation of characteristics with time. Based on its evaluation of the water processing element developed by the Airesearch Division of Garrett Corporation for the Army's MUST field hospital, the U. S. Army Environmental Hygiene Agency (Ref. 67) concluded that the 595-gallon equalization tank provided virtually no equalization. They suggested that equalization based on the 24-hour flow would be preferable. Whether 24-hour equalization is necessary at a bare base is a matter for further research. It will be assumed that 24 hours is necessary but this assumption is subject to revision pending the necessary research.

A waste detention time of 24 hours in the equalization tank will lead to septicity and odor problems unless preventive steps are taken. Aeration of the equalization tank would keep the contents of the tank aerobic and would provide the mixing required to make the waste characteristics homogeneous. If the temperature of the waste is in a range suitable for microorganisms, significant amounts of biological degradation could also take place in this basin.

e. Settleable Solids Removal

A settleable solids removal step is not included in the pretreatment scheme for either biological or physico-chemical treatment. Commercially available package waste treatment plants rarely include a settleable solids removal step prior to biological treatment. Its elimination simplifies the treatment system but it does require that more air be supplied to oxidize the added BOD in the biological processes. Since primary sedimentation sludge is easier to dewater than waste biological sludge, an added disadvantage is the replacement of primary sedimentation sludge by a sludge which is harder to handle.

Research on physico-chemical treatment methods has indicated that the settleable solids removal step can be eliminated as well (Ref. 66), thus improving simplicity of operation. Exclusion of this step is considered to have little effect on the coagulant dosage required for chemical clarification.

## 2. BIOLOGICAL TREATMENT

Under favorable conditions of temperature, pH, nutrient concentrations and tolerable concentrations of inhibitors, microorganisms can continuously remove dissolved organic matter from solution by synthesizing it into new cell mass and by catalyzing its oxidation. Aerobic oxidation requires that oxygen be present; the end products are generally  $\text{CO}_2$  and  $\text{H}_2\text{O}$ . In an anaerobic environment, anaerobic microorganisms can substitute other substances for oxygen in metabolic processes and produce partially oxidized end products such as methane, fatty acids, alcohols, aldehydes and other compounds as end products.

Microorganisms cannot significantly reduce the concentration of organic matter in water unless a sufficiently large number of organisms are present which have the particular characteristics of being able to promote oxidation of the specific compounds in the waste. This fact necessitates that sizable periods of time be allotted to start up a biological process; during this time the necessary number and type of organisms are developed.

### a. Activated Sludge Systems

Flocculated growths of microorganisms are continuously recirculated and contacted with organic waste in the presence of oxygen in the activated sludge process. Oxygen is supplied from air bubbled into the sludge-liquid



mass or by mechanical aeration. The process involves an aeration step followed by solid-liquid separation. A portion of the separated sludge is recycled as microbial seed and the remaining portion is disposed of by digestion, landfill, incineration or other suitable means.

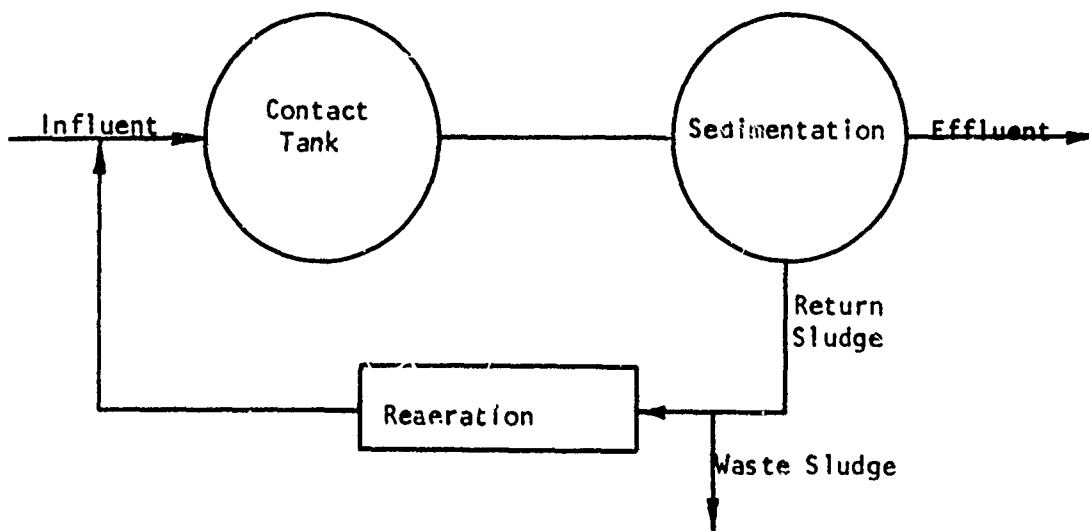
Many variations of the activated sludge process are employed in treatment of wastewaters. Commercially available package plants generally utilize the contact stabilization, completely mixed or extended aeration modifications, each of which are presented in Fig. 5. In contact stabilization the waste is thoroughly mixed with microorganisms for 30 minutes to 3 hours in the contact tank. The organisms are then removed by sedimentation. After their removal a portion is transferred to sludge treatment facilities and the remainder are stabilized by aeration for 5 to 6 hours in the reaeration chamber and then returned to the contact tank.

The completely mixed process involves maintenance of a homogeneous mixture of waste and microorganisms in the aeration basin. Contact time in this basis is 4 to 6 hours. Subsequent to aeration the organisms are separated in the clarifier; excess organisms are transferred to the sludge treatment process and the remainder are recirculated for reuse.

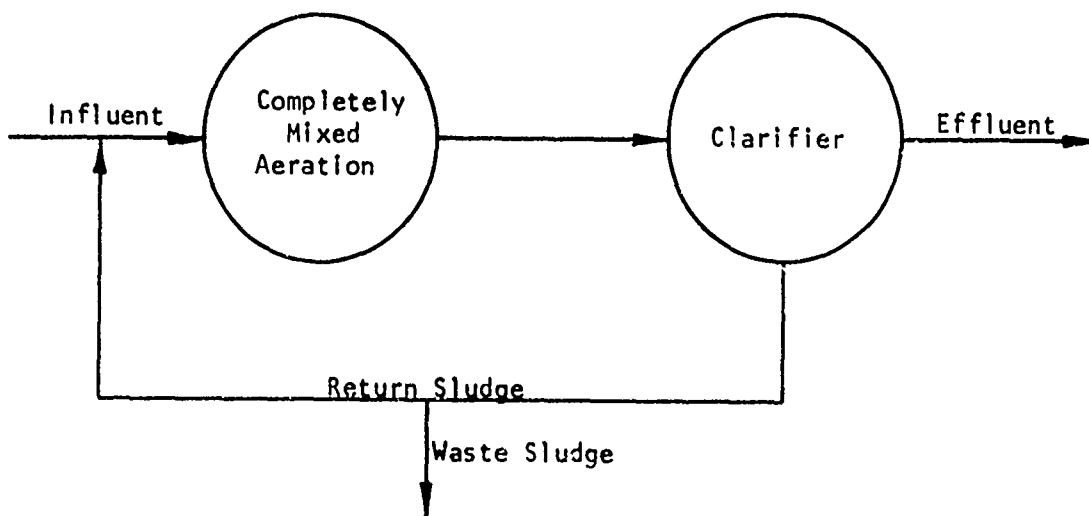
Extended aeration differs from the completely mixed process only in that the aeration period is on the order of 24 hours.

Design and performance data for activated sludge processes are given in Table 11.

An extensive evaluation of commercially available, packaged activated sludge plants is being undertaken in a later phase of the contract between the USAF and the University of Illinois under which this report was prepared. This evaluation is to be submitted in March, 1972, and includes



a. Contact Stabilization



b. Completely Mixed Activated Sludge and Extended Aeration

Figure 5. Modifications of the Activated Sludge System

detailed information on plants of potential use in the Mobility Program. Accordingly, no attempt is made to present detailed data on cost, operating characteristics, etc. in this report.

TABLE 11  
ACTIVATED SLUDGE DESIGN AND PERFORMANCE DATA\*

| Process Modification                           | Process Loading<br>#BOD/#MLVSS** | Hydraulic Loading<br>#BOD/1000 ft <sup>3</sup><br>day | Efficiency<br>% |
|--|----------------------------------|---|-----------------|
| Conventional Activated Sludge                  | 0.2 - 0.5                        | 35  | 95+             |
| Extended Aeration                              | 0.05 - 0.2                       | 20  | 75 - 90         |
| Contact Stabilization                          | 0.2 - 0.5                        | 70  | 85 - 90         |
| Completely Mixed Conventional Activated Sludge | 0.2 - 0.5                        | 35  | - 95            |

\* Ref. 112

\*\* Mixed Liquor Volatile Suspended Solids

In consideration of biological treatment, a study by Battelle Memorial Institute for the Naval Civil Engineering Laboratory (NCEL) (Ref. 12) merits examination. An evaluation was made of commercially available waste treatment plants with 32,000 to 65,000 gpd capacity. Waste characteristics included a BOD of 400 mg/l and suspended solids of 400 mg/l. A cost effectiveness study was performed which included among other considerations total cost, operation, erection and transportation. Based on this analysis they concluded that an extended aeration plant manufactured by Richards of Rockford Corporation was most cost-effective. This plant used a plastic lined earthen basin with mechanical aerators for the aeration part of the plant. This plant was subsequently purchased by NCEL and is currently in

operation at Point Magoo, California. Possible disadvantages in using this plant in the Mobility Program include the fact that the terrain may not be suitable for excavation for the aeration basin. Similarly, it may not be suitable for the Arctic because the earthen basin would be particularly difficult to heat and low efficiencies would probably result. Also, the start up period would be somewhat in excess of two weeks, similar to other biological plants.

The use of dried bacterial cultures has received some publicity in the past couple of years, particularly as a means of supplying microorganisms for controlling grease in wet wells, etc. Dried bacterial cultures are produced by Gerald C. Bower, Inc. of Orange, California. There is some speculation as to whether or not these cultures would serve to reduce the start-up time required for the typical activated sludge plant if large number these bacteria are introduced to the waste. It should be noted that proper operation of the activated sludge plant depends not only on the presence of a large number of microorganisms but also upon the presence of microorganisms which are acclimated to the particular waste characteristics. Unless the bacteria were grown on the specific type of waste under the specific conditions which will prevail at a particular base, the dried bacterial cultures would probably have little effect on reducing start-up time.

#### b. Fixed-Film Reactors

Fixed-film reactors comprise those biological processes wherein the microorganisms are firmly attached to solid media and the wastewater is brought into contact with these organisms. The trickling filter is one such example. Trickling filters are generally less efficient than activated sludge systems and because of this their usage has been less

frequent in municipal treatment systems in recent years. They are, however, easier to operate than the traditional activated sludge system. The efficiencies of trickling filters in removing biologically degradable organic matter commonly fall in the range of 65 to 85 percent. Design data are given in Table 12.

TABLE 12  
DESIGN DATA FOR TRICKLING FILTERS\*

| Type              | Hydraulic Loading<br>gpd/ft <sup>2</sup> | Organic Loading<br>lbs BOD <sub>5</sub> /1000 ft <sup>3</sup> /day |
|-------------------|--|--|
| Low Rate Filters  | 25 - 100                                 | 5 - 25   |
| High Rate Filters | 200 - 1000                               | 25 - 300   |

\* Ref. 68

A modification of the trickling filter is the rotating biological contactor (Bio-Disc) system manufactured by Autotrol Corporation. It consists of thin discs up to 11 ft in diameter spaced 1/2 to 1 1/2 in. apart on a shaft. The discs, made of high density styrofoam or other similar substances, are partially submerged and continuously rotated in a contact tank. Bacteria attach themselves to the discs that rotate in a direction opposite to that of the wastewater flow. Organic compounds adsorb on the microorganisms on the submerged portion of the disc and oxygen is supplied for biological oxidation on the portion of the disc exposed to the air. The power requirements for this system are considerably less than those for activated sludge because power is only required to rotate the discs. Torpey et al. (Ref. 113) used up to 10 stages of discs to determine removal efficiency as a function of the number of stages. Theoretical detention was 5 to 6 min. for each sequential stage and the

rotation was 10 rpm. For an influent BOD of 124 mg/l, 89 percent of this BOD was removed by the first seven stages. Suspended solids were similarly reduced from 107 to 14 mg/l. The subsequent 3 stages gave very small removals. A clarifier was required to remove excess microorganisms which sloughed off the discs. Growth was more rapid in the first 3 stages and had to be removed every 4 to 5 days to prevent bridging and anaerobic conditions between the discs (Ref. 113). Good data on the length of time required to build up active growth on these biological discs are not available but it is expected that a period of time similar to that required for trickling filters would be necessary.

#### c. Oxidation Ponds

Oxidation ponds consist of large earthen basins which may be lined with plastic liners to prevent contamination of the ground water. Liquid wastes are placed in these basins with essentially no pretreatment. A major effort is required to construct the basin but little or no mechanical equipment or power is required to operate the system. Oxygen for biological oxidation is supplied by natural aeration through the air-water interface. The organic matter which settles to the bottom of the lagoon is normally oxidized by anaerobic organisms. Oxidation ponds require essentially no maintenance and they work particularly well in suitable climates. The ponds are usually 2 to 4 ft deep with a detention time of 25 to 40 days. Organic loading is generally in the range of 30 to 50 lbs BOD/acre/day with the 30 lbs or less being used for cold climates. In cold regions, the ponds tend to freeze and this essentially stops biological action. The spring thaw brings about a nuisance period which lasts a few weeks (Refs 2, 3). The major disadvantages of oxidation ponds are poor operation in cold

weather, difficulties involved in restoring the site to its original condition after a period of use and problems involved in excavation in some locations. Major advantages include equalization of the wastes, low first cost and simplicity of operation.

A modification of the oxidation pond involves the use of mechanical aerators to speed up the biological oxidation process. With the provision of a sufficient number of aerators, basins with much shorter detention times and much higher organic loadings can be utilized. Depending on how much air is supplied and whether clarification of effluent with recirculation of microorganisms is practiced, the aerated oxidation pond can be considered an activated sludge plant. For example, the Richards of Rockford plant discussed above uses lined earthen basins for aeration and clarification with sludge recycle. If sludge return is not practiced with aerated oxidation ponds, the organic loading is intermediate between the non-aerated oxidation pond and the extended aeration sewage treatment plant.

#### d. Sludge from Biological Processes

When activated sludge package plants are used the general procedure is to use aerated digestion procedures to stabilize the sludge mass. It then can be dried on sludge drying beds and disposed of on land. Other procedures for sludge treatment and disposal are discussed in Section X.

### 3. PHYSICO-CHEMICAL TREATMENT

As shown in Fig. 4, physico-chemical treatment involves processes such as chemical clarification, filtration, primary sedimentation and ultrafiltration for suspended solids removal, and soluble organic removal by processes such as adsorption, chemical oxidation and reverse osmosis. Reverse osmosis will remove inorganic salts in addition to organic

compounds but adsorption and chemical oxidation will not. For the combined waste, reduction of total dissolved solids prior to disposal to the environment is not considered a necessity.

#### a. Chemical Clarification

The alternatives which should be considered in an evaluation of chemical clarification are given in Fig. 6. These include coagulant addition by electrochemical treatment or by hydrolyzing salts, flocculation and solids removal via sedimentation, upflow clarification or flotation. Polyelectrolyte may be used as a coagulant aid.

Chemical clarification normally removes up to 95 percent of the suspended solids and up to 70 percent of the  $BOD_5$  (Refs. 15, 16). Clarified waste can be applied to filters such as the multi-media filter or it can be applied directly to activated carbon adsorbers, in some instances, for the conjunctive removal of dissolved organics and suspended solids.

##### (1) Type of Coagulant

Chemicals such as lime, ferric sulfate, ferric chloride, aluminum sulfate, sodium aluminate, the ferrous salts and others have been used by researchers for coagulating suspended and colloidal solids in wastewater. In many instances the use of these coagulants has been aided by polyelectrolyte addition. Of those mentioned, ferric salts, aluminum salts and lime are most frequently used in wastewater treatment. These salts are all excellent phosphorus precipitants. There are certain advantages each has with respect to the others, however, and these advantages are reviewed in the following paragraphs.

Commercial lime is available as approximately 82 to 99 percent  $Ca(OH)_2$ . It can be purchased as  $Ca(OH)_2$ , or as  $CaO$ , in which case it is slaked prior to use as a coagulant. Lime has been used in the past for



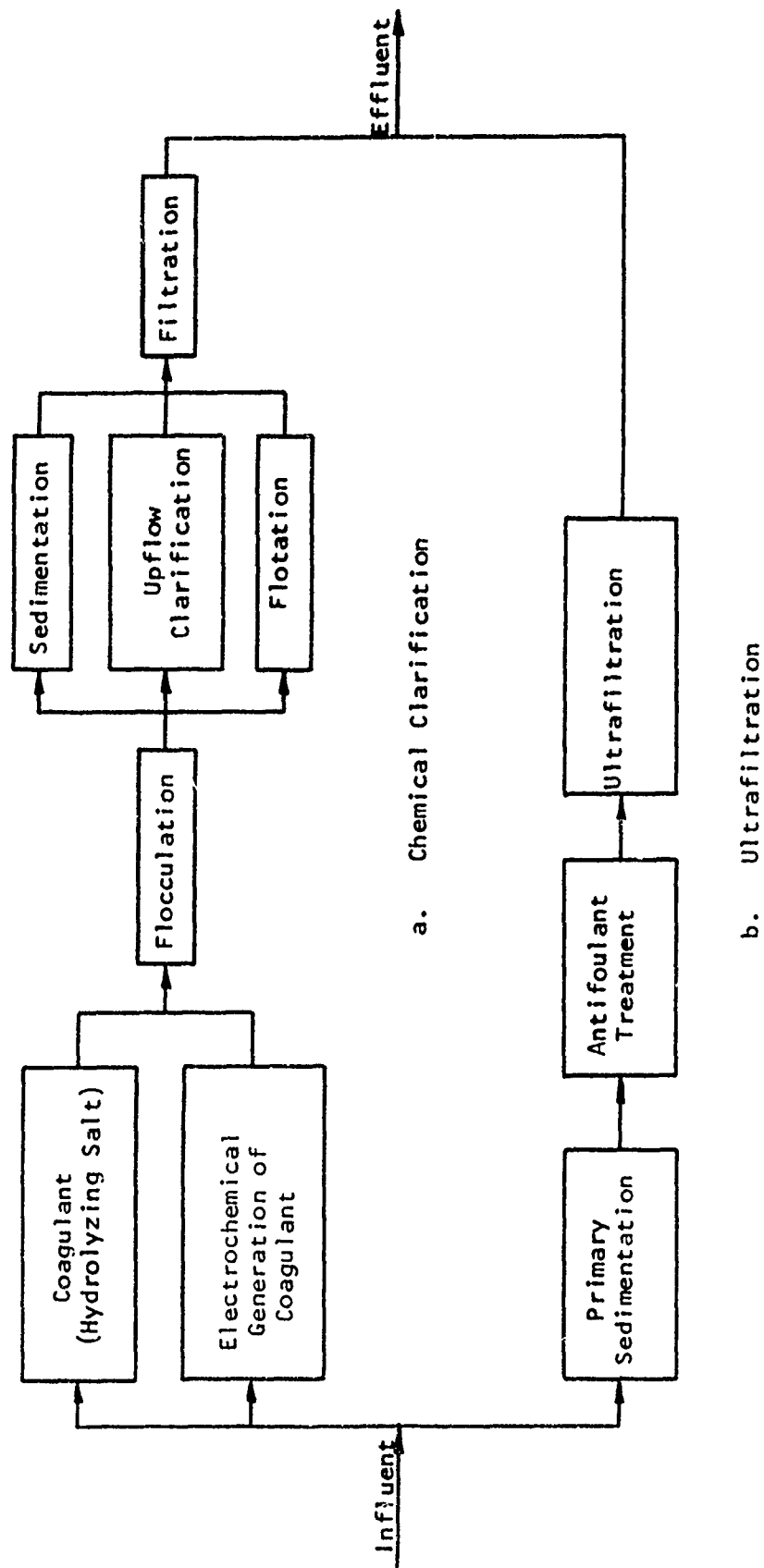


Figure 6. Alternatives for Suspended Solids Removal

large scale tertiary treatment at Lake Tahoe, in Windhoek, South Africa (Ref. 102) and in various pilot plants. One of the advantages of using lime is that the sludge may be thickened, dewatered and calcined to convert the calcium carbonate in the sludge to  $\text{CaO}$  which can then be reused. Extensive equipment is required for this process, however, and it appears to be economical only in plants with flows greater than about 10 mgd (Ref. 114). The process also has not been tested on lime-coagulated raw sewage. The cost of lime recovery when practiced is comparable to that of purchasing new lime, but savings in sludge disposal costs may be significant (Ref. 114).

If flotation is to be used to separate coagulated solids from wastewater, lime may not be desirable as a coagulant. In Windhoek, South Africa (Ref. 129) lime is used in conjunction with flotation. There are indications that satisfactory removals result only because of the high magnesium content of the water. The magnesium precipitates as magnesium hydroxide and makes a floc which apparently adheres well to air bubbles. Since a high magnesium content cannot be relied upon in every case it is not advisable to use lime in conjunction with flotation for the Mobility Program.

Ferric chloride and ferric sulfate have frequently been used for treatment of wastewater. These coagulants function over a wide pH range (3.5 and up) which is an advantage with respect to aluminum salts. The dosages range from 100 to 300 mg/l for raw waste water as compared with a typical lime dosage in the range of 300 to 600 mg/l (Ref. 14). Low dosages, 1 to 10 mg/l, of organic polymers may improve efficiency in solids removal and permit reduction of these coagulant dosages. The ferric salts do produce a denser floc than aluminum salts (Ref. 114). Ferric chloride is available in granular form containing up to 97 percent  $\text{FeCl}_3$ . This compound

is very hygroscopic and dissolves readily in preparation for feeding. Ferric sulfate is available in commercial form from 68 to 76 percent  $\text{Fe}_2(\text{SO}_4)_3$ . Ferric sulfate is difficult to dissolve, however, and ferric chloride may be the choice over ferric sulfate for this reason.

Aluminum is also frequently used as a coagulant. Aluminum sulfate is available as commercial alum with a minimum  $\text{Al}_2\text{O}_3$  content of 17 percent. Alum is usually more economical to use than the ferric salts and is thus used by a majority of water and wastewater treatment plants (Ref. 114). A disadvantage, if transportation over long distances is required, is that 13 to 16 waters of hydration accompany each  $\text{Al}_2\text{O}_3$  and thus raise transportation costs. The pH range of best coagulation is generally 5.5 to 8.5.

Coagulant can also be generated electrochemically. Pollution Engineering International, Inc. of Houston, Texas advertises an electrochemical process whereby aluminum and carbon electrodes are used and the aluminum is added to solution by electrolysis. In the solution it precipitates as aluminum hydroxide and performs the same function as alum. Coagulant is generated by applying a DC current source to ionize the aluminum. The electrodes are in parallel groups spaced from 1/2 to 1 in. apart, depending upon the water conductivity and design requirements. Foam is generated in the electrode tank and therefore a device for foam removal is required.

Both iron and aluminum electrodes were examined in a study of electrochemical removal of phosphates from wastewater. A cost of about 2.5¢/1000 gal and 8¢/1000 gal was found for iron and aluminum in a 1 mgd plant (Ref. 119). These costs exclude labor and filtration. Hydrogen generated in the electrolysis process did float some of the suspended solids in the

raw sewage simultaneously with phosphate removal. A potential of 5 volts and a current of .14 amps for 1/2 hr was used to reduce phosphate concentration from 40.5 to 0.2 mg/l.

An analysis of batch tests showed that flotation by the hydrogen generated at the cathode reduced suspended solids from 62 to 7.3 mg/l and TOC from 48 to 20 mg/l (Ref. 119). Extensive tests were not done on flotation, however.

A report by H. C. Miller and W. Knipe (Ref. 120) points out that a considerable amount of work was done on electrolytic wastewater treatment from 1900 to 1930. Several electrolytic municipal waste treatment plants were established but all were eventually abandoned for reasons of high cost of operation and questionable efficiency. The low electrical conductivity of primary and secondary effluents requires very close spacing between anode and cathode to electrolyze such solutions. Wastewater mixed with sea water is now being electrolyzed at two European plants with satisfactory separation of solids at a cost of about 6¢/1000 gal with a power cost of 1¢/kw-hr.

General Electric Company has designed and tested a 5780 gpd treatment plant for use on shipboard (Ref. 121). The plant includes an electrode system for addition of iron, as  $\text{Fe}^{+2}$ , to coagulate suspended solids. The electrocoagulation cell consists of parallel iron plates in a vented housing. A DC current is applied to alternate iron plates. The  $\text{Fe}^{+2}$  and negatively charged sewage form a very fine floc which requires that additional coagulant (50 to 100 mg/l of sodium aluminate) be added for its removal. The applied voltage is selected to provide an average ferrous iron concentration of 75 ppm in the wastewater. Air flushing nozzles were used to keep sludge from the cell plates but these were not

entirely effective. Test results showed that initial  $BOD_5$ 's of 75 to 200 mg/l were reduced to 48 mg/l, initial suspended solids of 600 to 1000 mg/l were reduced to 10 mg/l and turbidity was reduced from 70 to 5 JTU. One mg/l of iron was contained in the effluent from the plant.

## (2) Suspended Solids Removal Reactors

In chemical clarification, wastewaters are dosed with coagulation chemicals and flocculated; the suspended solids are then removed by either settling or flotation. Up to 95 percent of the total suspended solids and phosphorous can be removed by a well designed chemical clarification process. Alternative procedures for solids removal include the use of 1) upflow solids contact units, 2) independent rapid mix, flocculation and sedimentation units and 3) dissolved air flotation. A discussion of each of these alternatives as it relates to the present study ensues.

### (a) Upflow Solids Contact Clarifiers

Upflow clarifiers combine rapid mixing of coagulants, flocculation and solids separation in a single, compact basin. These units have found much application in water treatment in particular. Particles in the incoming flow are destabilized through chemical addition. The flow then passes upward through the flocculation zone, or sludge blanket, where the destabilized particles come in contact with other destabilized particles. As the particles increase in size, they settle into the sludge zone where they can be removed in a concentrated form. Sludge withdrawal from a solids contact unit should be at the same rate that it is being accumulated. Means for concentrating the sludge after its withdrawal should be provided because it is generally quite dilute. The clarification area should be sized on the basis of the settling velocities of the particles to be kept in suspension (Ref. 69).

Several investigators have reported on the evaluation of pilot contact clarifier equipment for the chemical clarification of sewage. Generally overflow rates of about 700 gpd/ft<sup>2</sup> were recommended for alum coagulated sewage. Alum floc was described as light and fragile; polymers were required to aid settling to prevent loss of floc in the effluent (Ref. 70). Sludge removal rates of 3 to 5 percent of the throughput were generally required to prevent septicity. The experiences encountered by Cohen and Kreissl (Ref. 71) in evaluating the Met-Pro system at Cincinnati, Ohio, and by the Environmental Control Technology Corporation of Ann Arbor, Michigan, (Ref. 80) in an evaluation of a physico-chemical system at Owosso, Michigan, indicate that floc from both alum and iron coagulated sewage is relatively unstable and subject to solids carryover.

Weber et al. (Ref. 66) reported successful operations at 700 gpd/ft<sup>2</sup> for separation of floc from lime-coagulated raw sewage. Data presented by Kalinski (Ref. 70) and Duff et al. (Ref. 70) indicate that higher overflow rates are permissible when using lime rather than alum. They suggest a design rate of 1800 gpd/ft<sup>2</sup> when using lime and 1200 gpd/ft<sup>2</sup> when using alum for the coagulation of secondary effluents. Polymer dosages in the order of 0.25 mg/l to aid settling were also recommended. A rate of 1400 gpd/ft<sup>2</sup> was forwarded for lime coagulated raw sewage (Ref. 70).

In summary, the advantages of using a contact clarifier in sewage treatment are: 1) it is a compact package unit performing all the necessary functions for chemical clarification, 2) short circuiting is minimized and 3) space requirements are small; the disadvantages are 1) higher operation and maintenance costs due to the relative inaccessibility of many parts, 2) more dilute sludges are produced, 3) a potential for higher solids

carryover exists, 4) skilled operation is required and 5) potential septicity exists in the sludge blanket.

(b) Independent Rapid Mix, Flocculation and Sedimentation

Rapid mixing basins for dispersion of the coagulants are usually equipped with high speed mixing devices designed to create velocity gradients of 300 fps/ft or more with detention times of 15 to 60 seconds. Power requirements for mechanical mixers are 0.25 to 1 hp/mgd (Ref. 70).

The basic equipment for flocculation is essentially the same as that used for flocculation in water treatment. The major difference is that detention periods in the flocculation basin may be reduced substantially in wastewater applications in which lime is used as the coagulant. Whereas flocculator detention times of 15 to 60 minutes have commonly been used in water treatment plant design, experience at the South Lake Tahoe plant has shown that flocculation times as low as 5 minutes may be satisfactory with lime coagulation (Ref. 70). The velocity gradients necessary for flocculation may be induced by mechanical means such as revolving paddles or by air diffusion. The velocity gradient,  $G$ , commonly employed in design is in the order of 30 to 100 fps/ft.  $Gt$  values used in design vary from  $10^4$  to  $10^5$  where  $t$  is the detention time.

Sedimentation basins are of many different types. Design overflow rates vary from 500 to 2000 gpd/ft<sup>2</sup> and detention times are as high as 4 hours. The use of tube or Lamella settlers allows the use of overflow rates 2 to 4 times those used in conventional settlers in some cases (Refs. 3, 6). These settlers are compact but are expensive and require more maintenance than conventional clarifiers due to the relative inaccessibility of many parts and the potential for sludge to adhere to the tubes.

### (c) Dissolved Air Flotation

Flotation of solids is accomplished by the introduction of microscopic air bubbles into the wastewater. As these bubbles rise, particles adsorb at the air-liquid interface and are thus carried to the surface where they can be removed. Bubble diameters range from 70 to 90 microns. Bubbles can be introduced by diffusers, or by saturating a portion of the waste flow with air and then permitting this air to be released from the solution by reducing the pressure in the flotation tank. Either influent or recycled effluent is pressurized in the presence of air to 25 to 50 psig. A retention time of 30 to 60 seconds is used in the pressure tank. Flotation is generally preceded by flocculation to promote particle growth.

Separation of suspended solids by flotation does not depend on the size and relative density of the particles as much as it does on their structure, surface properties and quantity of air used in flotation. For this reason, laboratory tests must be performed on the specific waste to be treated to determine design criteria (Ref. 72).

Factors of greatest importance in the design of a pressure flotation system are 1) feed solid concentration, 2) quantity of air used and 3) retention time if thickening of the float is required (Ref. 72). Past experience indicates an overflow rate range of 1 to 4 gal/ft<sup>2</sup>-min (1440 to 5750 gpd/ft<sup>2</sup>), a depth of approximately 6 ft or more to minimize turbulence and short circuiting, and a retention time of 10 to 40 minutes is required (Ref. 72).

A flotation unit which would meet these criteria would include the following basic elements: 1) pressurized pump, 2) air injection facilities, 3) retention tank or contact vessel, 4) back-pressure regulating device, 5) rapid mix and flocculation units, 6) chemical feeders and 7) a flotation chamber (Refs. 72, 79).



The advantages of dissolved air flotation are 1) reduction of septicity and associated odors owing to aeration of feed and much reduced detention times, 2) greater solids concentration in the float may be achieved than in gravity units, 3) greater efficiency in solids recovery in some cases, 4) more positive control over the process by controlling such variables as air/solids ratio and chemical addition, 5) smaller size and weight requirements, 6) removal of some of the dissolved organics by adsorption of surfactants on the air-water interface of the gas bubbles and 7) lower capital costs (Refs. 72, 74, 76). The fact that surfactants can be removed is particularly important in bare base operation since laundry and washrack wastes have a high detergent content and these wastes will be treated with the combined wastes.

The disadvantages include 1) higher operation and maintenance costs, 2) a requirement for more highly skilled personnel for operation, 3) the possible need to remove solids from the bottom of the tank as well as the top and 4) lower efficiencies of removal in some cases (Refs. 72, 74, 76).

#### b. Filtration

As shown in Fig 6, chemical clarification should be followed by filtration for reliable suspended solids removal. The filter would remove solids which escape sedimentation or flotation. A discussion of alternative filtration processes as they relate to this study follow.

##### (1) Microscreening

A microscreen is a filtration device which removes particles by the screening mechanism. These mechanical filters consist of a rotary drum that revolves on a horizontal axis and whose peripheral surface is covered with a stainless steel fabric. The size of the openings in the screen and

the pattern of the weave determine its effectiveness in removing suspended solids. Influent enters the open end of the drum and filters through the fabric with the intercepted solids being retained on the inside of the fabric. Strained effluent, pumped under pressure through a series of spray nozzles extending the length of the drum continuously removes the retained solids at the top of the drum. The solids and wash water are collected in a central trough within the drum and are discharged through a hollow axle (Ref. 74). The volume of wash water varies from 3 to 5 percent of the product water. The head of water applied (usually limited to 6 in. or less) and the concentration and nature of the suspended solids in the influent determine the rate of flow through the microscreen. Typical rates are in the range of 2.5 to 10 gpm/ft<sup>2</sup> of filtering fabric.

The advantages of microscreening include 1) low initial capital cost, 2) ease of operation, 3) light weight and 4) low headloss. The disadvantages of microscreens are 1) incomplete solids removal (Ref. 74), 2) inability to cope with solids loading fluctuations (Ref. 74) and 3) fouling of the screen with grease (Ref. 70).

The type of solids applied to the microscreen in bare base operation would probably be highly variable. Past experience in the microscreening of chemically clarified wastewaters has not been encouraging. Lynam et al. (Ref. 70) reported that microscreens could not produce good quality water from alum-coagulated secondary effluent. For these types of waters, other filtration devices are likely to function more efficiently.

## (2) Media Filtration

Many types of media are available for use in a bed configuration to remove suspended solids from waters. These media include diatomaceous earth, sand, anthracite coal, synthetic media and others. The sand filter

has most commonly been used in the water treatment field. The rapid sand filter has performed very satisfactorily in removal of suspended solids from chemically clarified waters. Removal of solids by diatomaceous earth is primarily by the same screening mechanism which was operable in the case of a microscreen. Thus the surface must be continually renewed by addition of more diatomaceous earth. After use, diatomaceous earth is disposed of along with other residues from the water treatment process. The diatomaceous earth filter has proved satisfactory in producing a high quality effluent. The need for continuous addition of filter media is a disadvantage because this requires feeders and a supply of fresh diatomaceous earth. The rate of feed of filter media should be a function of solids loading on the filter. Since this loading will probably be quite variable it will be difficult to feed the media as needed. Other media can be continuously cleaned and reused and thus do not have the same problems of media addition.

One undesirable aspect of sand as a media is that the sand grades hydraulically during backwashing with the finest particles rising to the top of the bed. During the filtration operation, these fine media act more efficiently to collect the suspended solids; as a result this portion of the bed clogs before media at a lower depth in the bed are used. This creates an inefficient use of the filter bed, although satisfactory quality can be reached with such beds.

An ideal solution, at least in theory, to the above problem is to reverse the direction of flow in the filter. Upflow filtration has great potential because coarse-to-fine filtration is achieved with a single medium. The full depth of the sand is utilized for the retention of removed impurities rather than the top few inches as in downward

filtration. As a substantial amount of suspended matter in such a filter is removed in the coarse portion of the filtration medium where it has less influence on head loss, a reduction in head loss through the filtration process is achieved for a given amount of solids removal (Ref. 77).

Upflow filtration is not the panacea for all filtration problems, however, because of some serious operational problems. Two major problem areas are fluidization of the finer sand and lifting of the bed permitting particles to escape. Bed lifting causing breakthrough of solids may occur in an upflow filter when the weight of the bed above a given level becomes equal to the head loss developed above that level (Ref. 77). Some investigators have used a grid system placed just below the top of the sand in an effort to solve these problems. Others have increased the depth of filter media and reduced the filtration rates. According to Hamann and McKinney (Ref. 77) neither increasing filter media depth to 6 ft nor use of a steel grid at the top of the filter bed was entirely satisfactory.

The concept of the bi-flow filter has lately been receiving increasing attention as an alternative to upflow filtration. In such a filter, water is introduced to the filter at both the top and bottom of the bed and filtrate is collected at some intermediate level within the media by a suitable collection system. The pressure developed by the downflowing stream, 15 to 20 percent of the total flow, in the upper part of the filter bed prevents expansion of the media. It is anticipated that problems would be experienced with collection systems in this case, however.

In spite of its potential attractiveness, the incorporation of upflow filtration into the bare base wastewater treatment system is not recommended. The present state-of-the-art, especially where wastewater

treatment is concerned, is not developed to the point where such a unit would afford reliable and trouble-free operation. Since other units are available which can efficiently produce a good quality water, it is considered more advantageous to use other units.

An approach to increasing the effective depth of downflow filters is to use a dual media bed with a layer of coarse coal above a layer of fine sand. Within each of the layers, fine to coarse stratification still occurs but the distribution of pore size with depth is much improved. The coal is used to prevent surface blinding and fine sand is used to provide maximum solids removal (Ref. 70). The particles held in the relatively large voids of the coal bed potentially can be dislodged with sudden changes in flow; these solids can be captured by the fine layer of sand at the coal-sand interface preventing breakthrough but causing blinding of the sand (Ref. 74). Some of the problems caused by the abrupt change in pore size at the coal-sand interface can be eliminated by intermixing these two layers to a certain extent by judicious selection of media size.

It is possible to add a layer of media such as garnet to further improve the pore size distribution of the bed. The advantage of an additional layer is countered by increased complexity of backwash, etc., and is not recommended for this reason.

Few data are available on the filtration of chemically coagulated raw wastewater. Many of our assumptions, therefore, have to be inferred from the studies of filtration of chemically coagulated secondary effluent. Rates of up to  $5 \text{ gpm/ft}^2$  have been used in the multi-media filtration of chemically coagulated potable water supplies (Ref. 70). Filter effluents essentially free of suspended solids as evidenced by turbidities as low as

0.1 JTU can be expected. Solids capacity values for dual media filters range from 1 to 1.5 lbs/ft<sup>2</sup> depending on the type of solids, flow rate and depth of bed (Ref. 78). Backwash is initiated upon solids breakthrough or when the headloss through the filter is 8 to 20 ft (Refs. 70, 78). Washwater requirements are 2 to 5 percent of the plant throughput, and the typical duration of each backwash is 5 to 8 minutes (Ref. 70). Backwash rates of 15 to 20 gpm/ft<sup>2</sup> are generally necessary along with a surface wash or air-assisted backwash.

The Hydromation Filter Company of Livonia, Michigan uses a polymer resin medium in their filter which is chemically resistant to all but chlorinated hydrocarbon solvents. Problems have been experienced in backwashing plastic media because they have a tendency to collect air bubbles (Ref. 127). It is the opinion of the authors of this report that these media could well be important for the treatment of a large variety of industrial wastes, but would hold little advantage for the wastes encountered in base operations unless Hydromation Equipment were used. Hydromation markets a radial flow filter which does not use conventional backwashing. Rather, dirty media are pumped to a scrubbing compartment where they are washed with a highly turbulent flow of water. The cleaned media are then returned to the filtration compartment.

### (3) Pressure vs. Gravity Filtration

The two basic filter structures in use are pressure and gravity filters. The principles of filtration are identical in both cases. The advantages of pressure filtration in relation to the Air Mobility Program are 1) higher head is available than is practicable with gravity filtration, 2) effluent from a pressure filter can be passed through a downstream process without further pumping, 3) washwater requirements are reduced

because the pressure filter can operate at higher head loss (Ref. 70), and 4) pressure filter systems are claimed to be less costly in small and medium sized plants (Ref. 70).

The disadvantages of pressure filtration are the general disadvantages inherent with the use of any pressure system. The danger of short circuiting which is cited as a problem by many public health departments is virtually eliminated if the effluent turbidity is constantly monitored as would be the case in a modern, well designed system.

Filters generally are oriented such that the flow is downward through the media. The Hydromation Corporation of Livonia, Michigan manufactures a filter in which the influent is introduced at the center of the median and flows radially outward. In radial flow, velocity decreases as the water progresses in an outward direction thereby creating conditions under which more efficient removal of suspended solids is possible.

#### (1) Moving Bed Filtration

In an effort to overcome the problem of premature surface clogging of sand filters, Johns Manville Corporation, under contract with the Federal Water Quality Administration, has developed a new filtering device known as the moving bed filter (MBF). Particulate matter is removed as the water passes through the sand (0.6 to 0.8 mm) medium of a MBF. As the filter surface becomes clogged, the filter median is moved forward by means of a mechanical diaphragm and sheared off, thereby exposing a clean filter surface. The sand is collected, washed in a separate compartment and returned to the base of the bed. The unit is thus a form of a counter-current extraction device with sand being fed countercurrent to the flow

of water. The major advantage of the MBF is the renewable filter surface and efficient use of the entire volume of filtering medium for removal of suspended solids. Also, the unit does not have to be taken off stream for backwashing since, in theory, 1 percent of the filter is being backwashed 100 percent of the time as opposed to the conventional practice of backwashing 100 percent of the filter 1 percent of the time. The moving bed filter principle allows much higher solids loadings than permissible with a fixed sand bed.

Technical data released by the manufacturer indicate a maximum unit capacity of 250,000 gpd. The dimensions of the main unit are about 6 ft x 19 ft x 21.5 ft with the auxiliary units occupying a space of approximately the same area. The washwater requirements are listed at 7 1/2 percent of the influent flow and the motor requires 10 horsepower. The total weight of the unit is 78,000 lbs. No cost data are available at present.

The MBF does not appear to be developed to the point where an impartial evaluation and recommendation can be made on its use in the Bare Base Program. The capacity, size, and weight listed above are all high in relation to the Bare Base Program requirements; no doubt smaller units could be built once the development and testing stages are complete. The washwater requirements also appear to be high. Because of these uncertainties, this filter is not considered further in this report. This should not in any way foreclose another analysis of this filter at a future date.

#### c. Ultrafiltration

As shown in Fig. 6, ultrafiltration or hyperfiltration is a process which can be considered for removal of suspended solids from wastewater. It



should be preceded by pretreatment consisting of primary sedimentation for removal of settleable solids and possibly of antifoulant conditioning to improve operation. Ultrafiltration is a process similar to reverse osmosis except that membranes with large pores are used. The use of large pores enables operation at high flux rates as compared with reverse osmosis and also effects the removal of very large molecules and suspended solids. Most of the inorganic salts pass through the membrane.

Few data are available on this particular treatment process as applied to primary effluent; two reports have appeared, however, which permit a preliminary assessment to be made of the process (Refs. 124, 125). Oak Ridge National Laboratories studied the removal of organic compounds and inorganic salts by dynamically-formed reverse osmosis membranes (Ref. 124). Both primary effluent and secondary effluents were used in their studies. Rejection efficiencies depended very much on the characteristics of the membrane, but it was found possible to reject one-fourth to one-third of the dissolved inorganic salts and 80 to 90 percent of the COD at initial flux rates of 50 to 100 gpd/ft<sup>2</sup> and pressures of 450 to 1000 psi. Runs of several days were used. Unfortunately, suspended solids removal was not examined; apparently both the primary and secondary effluents had been clarified before use. A cost analysis was not made as part of the study. Since flux rates are much higher than in reverse osmosis, capital costs of the system would be less. Operating cost would probably be much higher because axial velocities of 10 to 44 fps had to be maintained to prevent clogging of the membrane. This would result in excessive energy requirements. At lower velocities high rates of flux decline were a problem. Unfortunately, the studies were apparently carried out without regard to percent recovery.

In the study conducted by Aerojet-General (Ref. 125) on the reverse osmosis treatment of primary effluent, it was found that the only way a high rate of flux could be sustained was to maintain axial velocities of 12.9 fps (Ref. 125). At lower velocities, membrane fouling occurred which was attributed primarily to the precipitation of calcium salts. Biological growths were a problem at pH values near neutrality but did not occur at pH values of 5.3 or 6.0.

Primary effluent feed water preconditioned by pH adjustment and addition of a solids deposition inhibitor was found to cause a very rapid flux dropoff for axial velocities of 2.8 and 6.45 fps. A test with an axial velocity of 12.9 fps resulted in a dramatic improvement in membrane performance but the test was marred by tubular structure failures and pumping failures. The flux declined from 27 to 14 gpd/ft<sup>2</sup> in 15 days (Ref. 125).

In summary, significant operational problems were encountered with both the high flux, high pressure membranes and the typical reverse osmosis membrane. Ultrafiltration would probably be affected similarly and that reliable operation will not be possible for several years. The large amount of concentration requiring disposal is also a serious problem and severely limits the application of ultrafiltration. In comparison with the other suspended solids removal operations, the collection of solids in the form of a sludge which can be thickened and then disposed of by incineration, etc. is a much better alternative because of the greatly reduced volume of waste and greatly reduced operational problems.

#### d. Removal of Dissolved Organic Compounds

Alternatives for removal of dissolved organic compounds are given in Fig. 7. These include adsorption onto activated carbon or synthetic

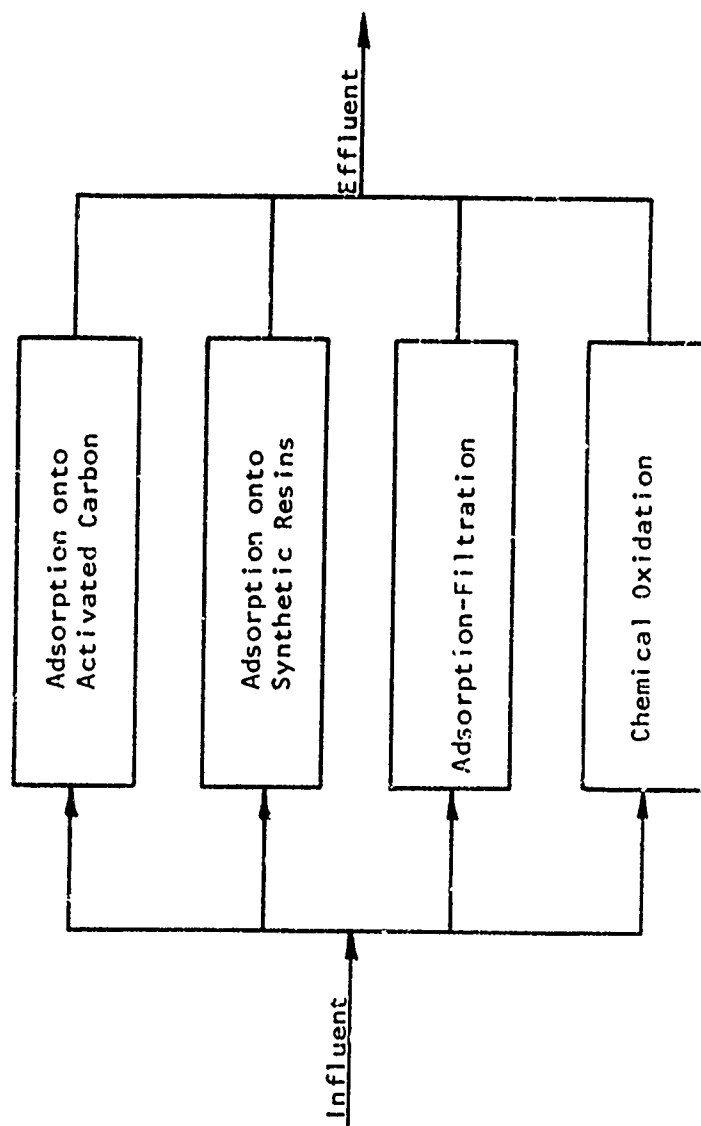


Figure 7. Alternatives for Removal of Dissolved Organic Compounds

resins and chemical oxidation. The use of an activated carbon adsorber for suspended solids removal in addition to adsorption is also a possibility. Conjunctive use of the adsorber in this manner would permit deletion of the filtration step which was discussed above.

#### (1) Adsorption onto Activated Carbon

Activated carbon is an adsorbent medium characterized by an extensive system of internal pores which provide it with a very large surface area per unit weight. This large area, coupled with the variety of functional groups attached to its surface gives activated carbon a substantial adsorptive capacity for dissolved organics in wastewater. Activated carbon in contact with water has a preference for nonpolar organic molecules. Treatment with activated carbon will remove many refractory substances as well as biodegradable compounds. The rate and extent of adsorption of most organics found in wastewater decreases with increasing pH and at pH values above 9.0 becomes very poor. When carbon is contacted with the wastewater, organics collect on the carbon's surface until its saturation capacity is reached. Saturated carbon can be thermally regenerated or discharged to waste. Partial regeneration can be accomplished using certain chemical reagents.

##### (a) Granular vs. Powdered Activated Carbon

The two general types of carbon adsorption systems are those utilizing granular carbon and powdered carbon. Owing to the fact that powdered activated carbon is not normally applied in a countercurrent fashion, its capacity for organic removal is not fully utilized; this leads to much higher dosage requirements and lower quality effluents. In a recent study, Battelle-Northwest (Ref. 81) determined an average powdered carbon dose of 800 mg/l to be necessary for a wastewater with considerably lower organic content than that expected for bare base

wastewater. This dosage is about 32 percent higher than the dose of granular carbon conservatively estimated for the treatment of bare base wastewaters.

Even though regeneration facilities are not anticipated at a bare bass [regeneration is not considered to be economical for municipal tertiary plants under 1 mgd (115)], the possibility of transporting the spent activated carbon for regeneration should not be overlooked. This could be effected at minimal cost if unused space and weight on returning Air Force transport planes are used for this purpose. Regeneration systems for granular activated carbon have been developed to a much greater extent than those for powdered activated carbon.

Unless strict procedures are enforced for handling and transporting powdered activated carbon, a severe dust and explosion hazard will be encountered.

Since activated carbon is insoluble in water and has a specific gravity greater than 1, the use of powdered activated carbon would necessitate the provision of storage tanks equipped with devices to keep the carbon slurry well agitated (Ref. 70). From a strictly operational point of view the use of granular activated carbon is cleaner and easier to apply than powdered activated carbon.

Powdered carbon may be more desirable in small installations where regeneration is out of the question from a purely economic perspective and powdered activated carbon becomes cheaper to use than granular activated carbon. An additional advantage of using powdered activated carbon is that it can be fed directly into the chemical clarification units along with other coagulants and chemicals. Thus a capital cost savings on carbon columns and the related support equipment is made possible. A filtration

step would probably be necessary after the powdered carbon contactor. It is not necessary to have a filtration step preceding it, however.

(b) Design Considerations for Granular Carbon Systems

In granular activated carbon systems the clarified, and in some cases filtered, wastewater is passed through a bed of granular activated carbon particles, usually 8 x 30-or 12 x 40-U. S. Standard mesh. Granular activated carbon systems are usually staged, and can be either upflow or downflow. Upflow beds can also be of the expanded or packed bed types. Packed bed columns play dual roles in that they also function as filters for the removal of suspended matter. They, therefore, are subject to clogging and need to be backwashed periodically.

The flow through activated carbon systems is usually counter-current meaning that the flow is contacted with progressively cleaner (less exhausted) activated carbon. Such an arrangement makes optimum use of the adsorptive capacity of the carbon.

A summary of data collected from the literature on activated carbon adsorption systems is given in Table 13.

TABLE 13  
ACTIVATED CARBON DESIGN DATA

| Parameter  | Range of Data             |
|--|---------------------------|
| A. C. Exhaustion Rate (lbs COD removed/lb A.C.)<br>(lbs A.C./M.G.) | 0.25 - 0.60<br>250 - 1000 |
| Number of Stages   | 1 - 4                     |
| Carbon Loss on Regeneration (percent)                              | 4 - 8                     |
| Hydraulic Loading (gpm/ft <sup>2</sup> )                           | 2 - 10                    |
| Backwash Rate (ft <sup>2</sup> )                                   | 15 - 18                   |
| Empty Bed Contact Time* (minutes)                                  | 15 - 64                   |
| Fuel Requirements for Regeneration (BTU/lb A.C. regenerated)       | 4250                      |

\* Empty Bed Contact Time is  $V/L$  where  $V$ =velocity of the water approaching the bed in ft/min and  $L$ =column length.

One of the most important design parameters for activated carbon systems is the contact time. At application rates between 4 and 10 gpm/ft<sup>2</sup>, the product quality is a function only of column length or residence time. The actual velocity of the water past the carbon granule in the range mentioned has no major effect on the rate of adsorption. Table 14 gives the total required length of activated carbon column as a function of overflow rate and empty bed contact time.

TABLE 14  
COLUMN LENGTH AS A FUNCTION OF OVERFLOW RATE  
AND CONTACT TIME FOR ACTIVATED CARBON SYSTEMS

| Empty Bed<br>Contact Time<br>(min) | Hydraulic Loading Rate (gpm/ft <sup>2</sup> ) |      |      |      |
|------------------------------------|---|------|------|------|
|                                    | 4   | 6    | 8    | 10   |
|                                    | (ft)  | (ft) | (ft) | (ft) |
| 10                                 | 5.3   | 8.0  | 10.7 | 13.4 |
| 20                                 | 10.7  | 16.0 | 21.4 | 26.7 |
| 30                                 | 16.1  | 24.1 | 32.1 | 40.1 |
| 40                                 | 21.4  | 32.1 | 42.8 | 53.5 |
| 50                                 | 26.7  | 40.1 | 53.5 | 66.9 |
| 60                                 | 32.1  | 48.2 | 64.2 | 80.2 |

The kinetics of the adsorption step depends on many factors including the diameter of the carbon granule, the temperature and the pH. As far as particle size is concerned, the finer the material, the faster the rate of adsorption.

A correlation between contact time and organic removal is not one that can be generalized and formulated. This relationship is highly dependent on the characteristics and properties of the particular waste stream under consideration. However, the information summarized in Table 15

was derived from the work of several investigators and does give an indication of the removals which can be expected. No temperatures are indicated, and varying degrees of biological activity within the columns, varying degrees of pretreatment, and waste differences all contribute to the differences observed.

Another important design parameter is the unit capacity of activated carbon for organic matter. This allows the determination of the exhaustion rate of the activated carbon and therefore the rate at which it must be regenerated or, alternatively, supplied. Again, no firm set of data is available but the information summarized in Table 16 was derived from the available literature. Different degrees of biological enhancement and other differences attributable to temperature, operating conditions, grades of carbon used, type of COD and other factors contribute to the observed variations in loading.

TABLE 15  
ORGANIC REMOVAL AS A FUNCTION OF CONTACT TIME

| Influent                   | Carbon Mesh Size | Contact Time (min) | COD Removal (percent) | TOC Removal (percent) | Reference |
|----------------------------|------------------|--------------------|-----------------------|-----------------------|-----------|
| Clarified Raw Sewage       | ?                | 33                 | 75*                   |                       | 75        |
| Secondary Effluent         | 8 x 30           | 40                 | 80                    |                       | 82        |
| Secondary Effluent         | 8 x 30           | 15                 | 55                    |                       | 70        |
| Secondary Effluent         | 16 x 40          | 40                 | 78                    |                       | 83        |
| Clarified Primary Effluent | 12 x 40          | 40                 |                       | 75**                  | 66        |
|                            | 12 x 40          | 20                 |                       | 62**                  | 66        |

\* Includes some suspended COD removal  
\*\* 10 percent expanded, fluidized beds



TABLE 16  
CAPACITY OF ACTIVATED CARBON FOR DISSOLVED ORGANIC MATTER

| Activated Carbon<br>Mesh Size | Unit Capacity of Carbon<br>(lb COD/lb carbon) | Reference |
|-------------------------------|---|-----------|
| 8 x 30                        | 0.50  | 70        |
| 8 x 30                        | 0.52  | 82        |
| 16 x 40                       | 0.58  | 83        |

According to Cover and Wood (Ref. 82), for a staged downflow arrangement, a two-stage adsorber is economically preferable because its carbon loading capacity is greater than a one-stage adsorber with similar operating cost, and its operating costs are lower than the operating costs of either three or four-stage adsorbers. The less exhausted of the two adsorbers is always second in line. When the full capacity of the first carbon column is utilized, its carbon is removed for either regeneration or disposal and fresh activated carbon is provided. This unit is then placed second in line.

Carbon column adsorbers are either downflow or upflow. An upflow packed-bed adsorber may be used as a "moving bed" adsorber. This technique is an application of the countercurrent operating principle in that the flow is upward through the bed, with spent carbon being periodically removed from the bottom of the vessel. Fresh carbon is added to the top of the column. This system accomplishes essentially full utilization of the adsorptive capacity of the carbon. However, its use must be associated with water containing a minimum amount of suspended solids (Ref. 84).

Expanded-bed upflow columns may be used with wastewaters containing suspended solids (Refs. 66, 70) but these waters would then require filtration subsequent to adsorption. These beds do not require backwashing. Sufficient velocities must be maintained, however, to expand the media by about 10 percent so that the bed will be self cleaning. An alternative to maintaining an expanded bed under low flow conditions is to use air to maintain fluidization. A bed originally expanded 10 percent will expand further when restricted by accumulated solids to continuously flush the solids through the bed and thus avoid increased headloss (Ref. 70). This means a flow of  $10 \text{ gpm/ft}^2$  at  $72^\circ\text{F}$  for 8 x 30-mesh carbon (Ref. 70) or  $5 \text{ gpm/ft}^2$  for 12 x 40-mesh activated carbon at the same temperature (Refs. 66, 70). The work of most investigators, including that of Weber et al. (Ref. 66) indicates that the effect of suspended solids on the efficiency and life of granular carbon is negligible. Among the advantages of upflow expanded columns are 1) reduction of headloss since the headloss across the carbon bed is equal to its weight in water and 2) elimination of plugging and clogging and backwashing equipment.

The disadvantage of expanded bed adsorbers include a requirement for greater column lengths for a given contact time if only water flow is used to keep the bed fluidized. If air is used to aid fluidization, lower flow rates are possible. Higher overflow rates require longer column lengths for the same contact time.

Two carbon sizes are normally utilized for the treatment of wastewaters, 8 x 30-U.S. Standard mesh and 12 x 40-mesh. The finer material (12 x 40-mesh) has a higher rate of adsorption, but for packed beds also has a higher headloss per unit depth of bed, and due to smaller interparticle voids, it has a greater tendency to plug with materials filtered out of the wastewater. In addition, regeneration losses at 10 percent (Ref. 82) are twice

those for the coarser media. However, where fluidized expanded beds are employed, the overflow rates for the coarser media are twice those required by the finer media for proper expansion of the bed. Therefore, it appears that where fluidized expanded beds with no regeneration are used, the 12 x 40-mesh carbon is preferable, while for packed beds with regeneration, the coarser 8 x 30-mesh carbon is preferable. Other arrangements (e.g. expanded beds with regeneration, etc) should be studied on a case by case basis.

At 72°F, the headloss during the carbon service cycle for packed beds utilizing 8 x 30-mesh carbon varies from about 2 in. of water per ft of carbon depth at 4 gpm/ft<sup>2</sup> to about 5 in. at 10 gpm/ft<sup>2</sup> for a clean bed. Headloss figures for the various grades of carbon have been published (Ref. 85). In expanded bed columns, the headloss equals the weight in water of the carbon.

Packed beds of activated carbon eventually clog up with suspended solids and need to be backwashed. This is done when the headloss across the bed reaches a predetermined level. The proper design of the backwash equipment is of the utmost importance and this includes the provision of surface wash appurtenances. Clean water is desirable for backwash purposes.

An important column design consideration is the depth to cross-sectional area ratio. With properly designed distribution and collection arrangements this ratio is not critical and values of even less than 1:1 may be employed. However, with poor distribution and collection arrangements, ratios of 2:1, 4:1, 10:1 or higher should be utilized to prevent short circuiting (Ref. 70).

## (2) Adsorption onto Synthetic Resins

A major disadvantage in using activated carbon is the fact that

many molecules adsorb very strongly and necessitate thermal regeneration or discarding of the carbon. Certain manufacturers are now manufacturing synthetic resins with various surface characteristics which can easily be regenerated without being removed from the adsorbent column. Regeneration can be accomplished by solvents such as caustic or methanol. The removal mechanism which appears to be of most importance is physical adsorption, but anion exchange and cation exchange may also be important for organic anions and cations, respectively (Ref. 128). In studies at the Sanitary Engineering Research Laboratory, University of California, (Refs. 99, 100) the activated sludge effluent which had been treated by alum coagulation, sedimentation and rapid sand filtration was treated by different types of resins. The best resin for COD removal was one which had a macroporous structure with amine functional groups. In repeated tests the resin showed satisfactory removal of refractory organics and complete reversibility using caustic. Color removal was essentially 100 percent. There was no loss in performance efficiency reported during nine exhaustion-regeneration cycles. Four lbs of 2 percent NaOH was used to regenerate 1 cu ft of resin. Comparative studies with activated carbon showed that adsorption was not reversible in the same manner, i.e. the adsorption efficiency declined from 100 percent for removing color in the first cycle to 60 percent in the sixth cycle. When the resin and carbon were used in series the two adsorbents complemented one another. Essentially 100 percent color removal and 80 percent COD removal was achieved on clarified secondary effluent. The cost was estimated at 7¢/1000 gal for resin plus activated carbon vs. 10¢/1000 gal for activated carbon alone.

Because of the very limited amount of work which has been done using resins as adsorbents, it cannot be stated with certainty that the

resin by itself, or a combination of resin and activated carbon will be an improvement over activated carbon utilized alone for bare base wastewaters. It appears to have good potential, however, and should be pursued because of the possibility of eliminating the need to supply large quantities of fresh carbon.

### (3) Adsorption-Filtration

Packed beds of activated carbon can serve as filters for removal of suspended solids in addition to removing dissolved organic matter. The use of carbon beds in the water purification industry has frequently been for both functions (Refs. 116, 117, 118). Combining the two processes into a single structure has an obvious advantage of eliminating one of the processes and potentially simplifying overall operation. Whether or not granular carbon beds can satisfactorily serve in this capacity in a physical-chemical treatment scheme has not satisfactorily been answered by research. There are indications that a longer backwashing time, i.e. 60 to 90 minutes instead of 20 minutes may be required as compared with the traditional media filter (Ref. 70). Because of the potential advantages that could develop in terms of making the unit more compact, it is felt that this aspect should be closely investigated in further research.

### (4) Chemical Oxidation

Chemical oxidation is an alternative procedure for removing dissolved organic substances from water. The only chemical oxidant which has potential application in this regard is ozone. Ozone has been used primarily for disinfection, taste, odor and color removal from municipal water supplies. In this respect its application has largely been in waters with much lower organic concentration than would be experienced in the waste treatment system on a bare base. Gardiner and Montgomery (Ref. 123)

studied the removal of organic compounds from trickling filter effluents. Ozone dosages were applied in the range of 10 to 50 mg/l to waters with COD near 40 mg/l (vs. expected COD of 400 - 500 mg/l for bare base waters). Approximately 1/2 mg/l COD was removed for each mg/l ozone applied. Typical results indicated 30 minutes of contact was necessary to reduce COD from 56 to 32 mg/l. Ozone absorbed was 41 mg/l. It is essential to note that for larger COD removals more extensive contact time and much larger ozone dosages are required.

In order to reduce COD from the 400 - 500 mg/l range to levels achievable with adsorption, extremely large doses of ozone and large reactors would be required. Other researchers studied COD removals from a clarified effluent which had a COD of 35 mg/l. They found that this 35 mg/l could be reduced to 15 mg/l at a cost of 7.7¢/1000 gal, assuming a high ozone utilization efficiency for a 10 mgd plant (Ref. 126).

#### e. Disinfection

An up-to-date summary of the state-of-the-art of disinfection has been given by J. C. Morris and his co-workers on the AWWA research committee on Disinfection and Chlorination (Ref. 122). Chlorine is an effective bactericide as is well known. Virus are somewhat more resistant to chlorine and, therefore, conditions under which chlorine functions best as a virucide are important. Morris reports work by others which showed that 0.5 to 1 mg/l  $\text{HOCl}$  produced satisfactory inactivation of enteric virus in 30 minutes; the same effect was achieved in one case by 25 mg/l chloramine and in another case by more than 100 mg/l of  $\text{OCl}^-$ . Since wastewaters contain ammonia, it is important that chlorine be applied to waters under conditions in which the rate of reaction to form chloramines is minimized. Work of Palin, as reported by Morris (Ref. 122) shows this to be the case at a pH of 7.5.

It is also essential that chlorine be rapidly mixed with the flow and that the chlorination reactor be of the plug-flow type to minimize short circuiting (Ref. 122).

Morris further reports that bromine and iodine have some desirable properties for disinfection (Ref. 122). Bromine exhibits chemistry similar to that of chlorine;  $\text{HOBr}$  and the bromamines appear to be as effective as  $\text{HOCl}$  and chloramine. Bactericidal and virucidal efficiencies have been found to be similar. The one difference is that bromine has its peak efficiency at about pH 8.5 as opposed to 7.5 for chlorine. In spite of bromine's advantages, it seems unlikely that it will prove to be effective as a substitute for chlorine because of its greater cost, scarcity, and possible physiological effects (Ref. 122). Thorough investigations on a plant scale to determine engineering problems likewise have not been performed. Iodine's greatest advantage is that it does not react with nitrogen type compounds.  $\text{I}_2$  is a good bactericide and cysticide while  $\text{HOI}$  is a good virucide. However, similar to bromine, iodine seems unlikely to become a municipal water supply disinfectant in any broad sense because of cost, availability and possibly physiological activity (Ref. 122). Its usefulness seems to be limited to noncontinuous type applications.

Ozone is one of the best possible alternatives to chlorine for disinfection (Ref. 122). Because of cost, engineering problems and the characteristics of ozone in solution, ozone has not been used as extensively as chlorine. It is popular in France and Switzerland, however. Ozone is not seriously hampered by the presence of nitrogen and is effective over a wide pH range; recently, engineering improvements have been made in the production of ozone and its application to water. Ozone is effective against viruses, bacteria and cysts and is also effective in destroying odor, taste and color

in water supplies. Its decomposition product is oxygen. A residual of 0.4 mg/l after 15 minute contact provides an efficiency equal to the best chlorination practice (Ref. 122).

A disadvantage of ozone is that it is not selective in its oxidizing action. Waters with large concentrations of organic matter will have high ozone demands thus increasing greatly the cost of its application. In the past ozone has most often been used for waters with low or constant demand (Ref. 122).

If chlorine is to be used, it can be obtained in cylinders as liquid  $\text{Cl}_2$ , in the form of salts such as  $\text{Ca}(\text{OCl})_2$  and it can be generated on-site by electrolysis of  $\text{NaCl}$ . Liquid chlorine cylinders may be hazardous under bare base operating conditions and are, therefore, not recommended.

Generation of chlorine on-site is possible. A non-eroding electrode system such as a lead dioxide anode and carbon cathode could be used to generate chlorine by electrolysis. A study in 1965 (Ref. 120) found that electrolysis of secondary effluent reduced a standard plate count from  $1.28 \times 10^6$  to  $3.6 \times 10^3$  after 15 minutes of electrolysis. Sizable reductions in COD, ABS and chloride occurred simultaneously; unfortunately a cost of \$1.21 to \$2.42/1000 gal was calculated. The capital cost for a 2 mgd plant was determined as \$2.7 to 5.26 million.

If  $\text{NaCl}$  were available at the bare base site, costs would be reduced. However, if  $\text{NaCl}$  must be shipped, on-site generation loses its attractiveness since  $\text{Ca}(\text{OCl})_2$  could be shipped as well.

Of the various salts which are available, calcium hypochlorite,  $\text{Ca}(\text{OCl})_2$  appears to be the most advantageous. Calcium hypochlorite can be supplied in the solid form in metal barrels or special lined drums. These



containers should be air-tight to prevent moisture from coming into contact with the hypochlorite. Contact with water or the atmosphere induces a pronounced increase in the decomposition rate of calcium hypochlorite. Additionally, hypochlorite containers should not be exposed to avoidable heat sources since the decomposition rate is increased by heat (Ref. 86). Commercial calcium hypochlorite should have at least 70 percent available chlorine. Based on an estimated dose of 20 mg/l as chlorine, the daily hypochlorite requirement equals approximately 8.3 lbs/1000-man increment.

#### 4. SELECTION OF THE RECOMMENDED TREATMENT PROCEDURE

##### a. Biological vs. Physico-Chemical Processes

The combined wastes can be treated by both the biological and physico-chemical process shown in Fig. 4. However, in spite of the wide applicability and reliability of biological treatment systems in municipal wastewater treatment, such systems contain inherent weaknesses insofar as the Mobility Program is concerned. The long start-up time required for activated sludge and fixed-film reactors (2 to 12 weeks) is a definite limitation in view of the 72-hour time constraint. The possible use of dried cultures of microorganisms for reducing the start-up time has been investigated but appears unlikely to be of much value.

Another weakness is the difficulty in operating biological systems on an intermittent basis. Microorganisms respond poorly to shock loading resulting from wide daily variation in quantity and characteristics.

A possible third weakness concerns the anticipated variation in climate. Rate of microbial activity decreases with decreasing temperature and, although the treatment system would undoubtedly be covered and heated

in very cold weather, the wastewater temperature would be lower causing slower rates of bacterial degradation of organics. Physico-chemical treatment processes are affected to a lesser extent by temperature and would thus be favored.

The start-up time limitation would not be as important if oxidation ponds were used. These ponds do operate poorly in winter, however. Restoration of site would also be a problem in that long periods of time are required for waste stabilization. It is also anticipated that some terrains will be encountered which will not be suitable for constructing oxidation ponds.

Physico-chemical processes can consistently achieve removal efficiencies comparable to, or in excess of, those for biological systems. These processes are not significantly affected by variations in parameters such as temperature, pH and organic and hydraulic loading, as are biological systems in general. They also do not require the start-up time for the build-up of the microbial sludge mass essential to biological systems and the 72-hour constraint should easily be attained. Systems based on physico-chemical processes require significantly less space than those based on biological processes.

In the event that reclamation of the wastewater is practiced, the effluent quality from the physico-chemical processes should be better than that provided by biological processes conventionally employed for secondary treatment. This statement must be qualified, however, in that certain biodegradable compounds may escape removal by chemical clarification and adsorption. This aspect requires close experimental evaluation.

The most critical constraint appears to be that of start-up time. Were this constraint to be significantly relaxed, biological processes should

be strongly considered. Commercially available prefabricated waste treatment plants, of which the majority are biological, are under evaluation at the University of Illinois as part of this same contract. The report on this evaluation is forthcoming in March 1972; many of the plants to be discussed in this report are suitable for use at a bare base.

b. The Recommended Processes

The recommended system for the combined wastes is presented in Fig. 8. The reasons for selecting this system are based on the analyses in the preceding portion of this section as well as additional factors presented below. Specific data on the selected processes and on alternative processes are presented below.

(1) Pretreatment

The pretreatment steps which are considered essential for good operation of the system are 1) size reduction and 2) equalization. Grit removal is not considered necessary because the washrack wastes will be degritted prior to transport and grit levels in other wastes are not considered as significant. In the event that grit removal were necessary an in-line degritter should be used because of its simplicity and efficiency. Such a degritter would cost approximately \$1000/1000-man increment.

Size reduction is necessary to prevent rags, etc. from interfering with equipment. An in-line grinder should be used to accomplish size reduction on the kitchen, dining room, shower and lavatory wastes which are piped to the treatment facility. Photographic washwaters, degritted and skimmed washrack wastes and non-human hospital wastes will be transported by truck to the central facility. It is anticipated that these trucked wastes can be discharged directly to the equalization basin without passing them through a grinder. The wastes to be transported by truck are

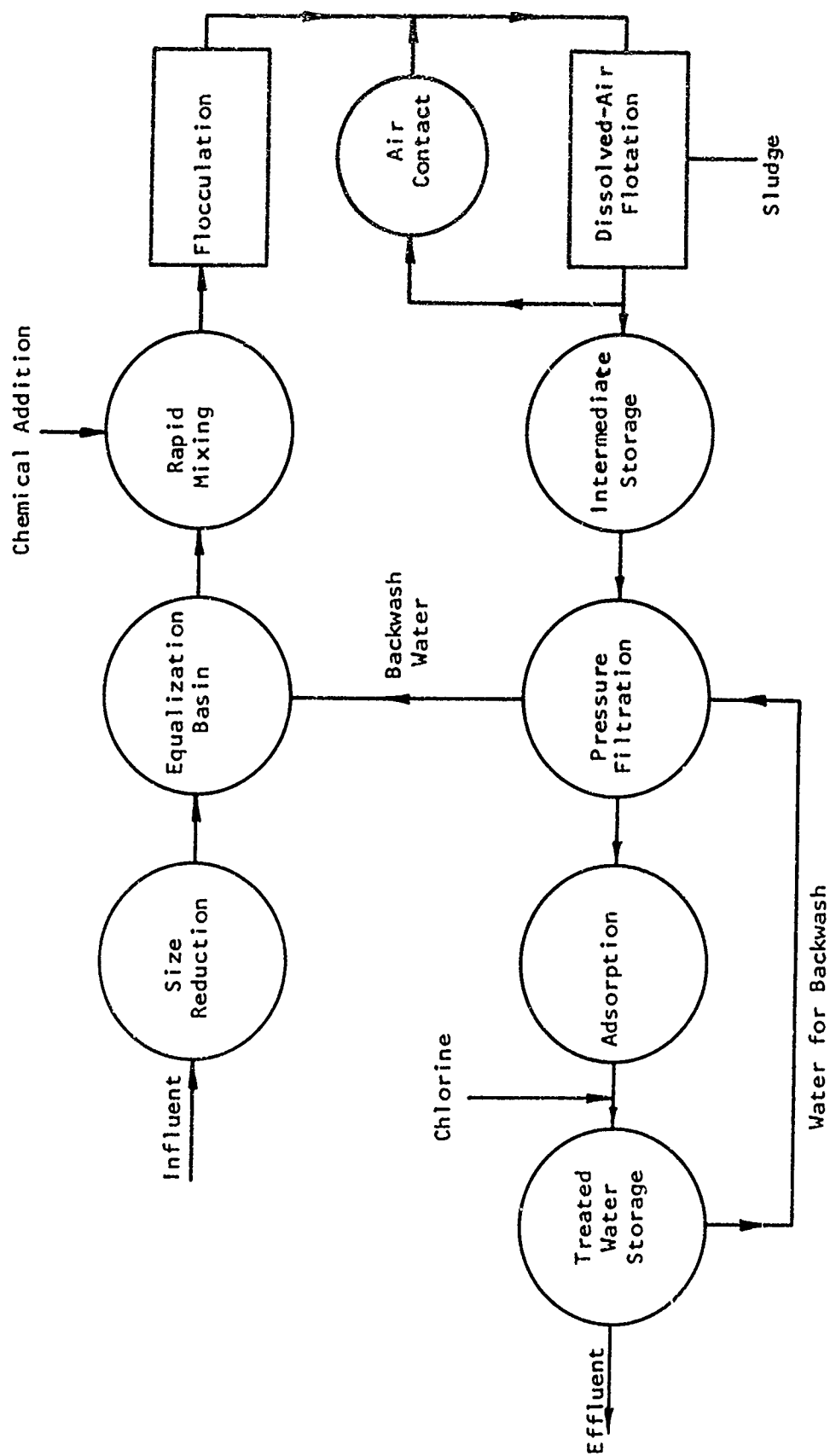


Figure 8. Schematic of the Waste Treatment System

stored in collection tanks prior to transportation allowing large solids to settle and thus making their exclusion possible. The cost of an in-line grinder is approximately \$500/1000-man increment.

An equalization basin with aeration to keep the waste fresh and to prevent sedimentation is required to eliminate upsets in treatment due to adverse changes in waste characteristics. The size of the equalization basin should be determined by further research. It is assumed that one day's capacity is required for the purposes of this report, however. A 35,000 gal, 3 in. redwood tank\* can be readily erected and dismantled on-site. It would have a shipping volume of 800 ft<sup>3</sup> and a shipping weight of 16,000 lbs/1000-man increment. The cost is approximately \$4500. Aeration and mixing equipment would cost an additional \$3500. If a smaller basin is shown to be satisfactory, savings in cost, shipping weight and shipping volume are possible. A 17,000 gal tank, for example, costs \$2000, weighs 6000 lbs and has a shipping volume of 306 ft<sup>3</sup>. Other sizes are also available. Galvanized steel or fiberglass could be used in place of wood, if necessary.

Aeration of the equalization basin, especially at the higher detention times, should produce biodegradation in many cases which would reduce the dissolved organic load on the plant. This could not be depended on in very cold climates but would occur in the majority of climates encountered. The extent of biodegradation is difficult to predict; however, compounds such as short-chain fatty acids which readily pass through adsorbers and reverse osmosis units would be the first to be degraded. Other materials may also be stabilized.

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\* Manufactured by Caldwell Tanks, Inc., Louisville, Kentucky.

## (2) Suspended Solids Removal

As shown in Fig. 6 the choice for a suspended solids removal process is between chemical clarification and ultrafiltration. The available data on ultrafiltration indicate that primary treatment should precede ultrafiltration to prevent abrasion of the membrane (Ref. 125). Even with primary sedimentation, significant problems have been encountered in ultrafiltration, and reverse osmosis, of primary effluent. Very high axial velocities are required to minimize fouling problems. The high velocities increase power consumption and cause equipment failures. Membrane life would probably also be shortened. Also long-term studies have not been carried out. Another significant factor is the large volume of residue which remains after treatment. An estimated 10 to 30 percent of the influent waste would require disposal as concentrate. This latter factor alone casts great doubt on the applicability of ultrafiltration in a situation where most residue will require incineration. It is felt that practical implementation of this process is several years from realization, and only then in areas where concentrate disposal is not a problem.

Chemical clarification is therefore the choice over ultrafiltration. It has been shown to be effective in clarifying many different types of wastewaters and should be effective on a bare base if the waste is equalized. A voluminous sludge is also produced but should be 3 to 5 times less than the concentrate produced from an ultrafiltration process operated at 90 percent recovery. Although a specific system has not been designed, it should be possible to meet essentially all of the Mobility Program criteria with a chemical clarification system.

In chemical clarification a choice must be made between the alternatives for coagulant addition. It can be added in the form of salts

or by electrochemical generation from an iron or aluminum electrode. A choice must also be made between sedimentation, upflow clarification and flotation. The type of coagulant, i.e. lime,  $Al^{+3}$ ,  $Fe^{+2}$ ,  $Fe^{+3}$ , depends to some extent on the reactor to be used for solids removal. Therefore, the type of reactor is discussed first.

In comparing the sequence of rapid mix, flocculation and sedimentation in three separate vessels with upflow clarification, in which all three processes can be carried out in the same reactor, an important advantage in savings of space is noted. It should be possible to obtain nearly the same efficiencies with each reactor type. A more dilute sludge is expected with upflow clarification and anaerobic conditions may develop in the sludge blanket. Upflow clarification is considered to be a more desirable choice because of its compactness in spite of potential problems, however.

Assuming a surface loading rate of  $800 \text{ gpd/ft}^2$ , which should be satisfactory for good removals, a surface area for the upflow clarifier of  $44 \text{ ft}^2$  is required. The clarifier would have a diameter of approximately 7.5 ft and a depth of about 10 ft. Such a clarifier should be available for approximately \$4500 to \$5000 (Ref. 108).

Upflow clarification should work satisfactorily for alum, lime and ferric chloride coagulated sewages. Potential problems may stem from the sludge blanket turning anaerobic because of too long a detention time, however. There also has been some indication of excess solids loss in the effluent.

Dissolved air flotation merits careful consideration because of the fact that much higher surface loading rates can be used. Flotation may also be able to remove considerable amounts of the surfactants which are in

the laundry, shower, lavatory and washrack wastes. The dissolved air flotation system consists of separate contactors for rapid mix, flocculation and flotation. Loadings in the flotation basin are generally on the order of 1500 to 5700 gpd/ft<sup>2</sup>. A retention time of 10 to 40 minutes is also required.

For 1000 men, a rapid mix tank of 24 gal is required for 35,000 gpd of flow and a detention time of 60 seconds. A mixing device that will create a velocity gradient of at least 300 fps/ft should also be provided. A flocculation tank capacity of 480 gal is required for a detention period of 20 minutes. A tank 8 ft long, 3 ft wide and 4 ft deep should be used. This will provide a 1 ft freeboard. The tank should be provided with 2 paddles designed and powered by a variable speed motor to permit tapered flocculation. The paddles should have a peripheral speed of 1 1/2 to 2 1/2 fps and should maintain velocity gradients between 30 and 100 fps/ft. Values of the dimensionless parameter, Gt (G is velocity gradient, t is detention time), should be 10<sup>4</sup> to 10<sup>5</sup>. The shipping volume and shipping weight of this tank are estimated at 100 ft<sup>3</sup> and 1400 lbs, respectively.

Research should be undertaken to determine whether influent to the flotation basin should be saturated with air, or if a portion of the flotation basin effluent should be recycled for pressurization and then mixed with the influent. Assuming that 25 percent of the return flow will be saturated with air at 25 to 50 psig pressure, a tank volume of 6 gal will provide the necessary contact time for air saturation. This tank should be capable of withstanding a working pressure of at least 50 psi. Air injection facilities and a pump capable of maintaining the design pressure should be provided.



Assuming 25 percent return flow, the total flow through the flotation basin is 30 gpm (43,250 gpd). The required surface area is  $21.6 \text{ ft}^2$  for a surface loading of  $2000 \text{ gpd/ft}^2$ . A tank 9 ft long, 2.5 ft wide and 7 ft deep is required. This tank should be provided with sludge removal equipment as well as float removal devices. The flotation unit and pressure vessel are estimated to have shipping volume and weight of  $450 \text{ ft}^3$  and 6000 lbs, respectively.

The capital costs of the rapid mix, flocculation and flotation equipment are estimated at \$15,000 to \$20,000/1000-man increment.

A net capital cost savings of \$10,000 to \$15,000 could be realized if upflow clarification was used in place of flotation. However, it is felt that flotation is a better choice because it can remove many surfactants thus lessening the load on the dissolved organic removal step. Other advantages include the elimination of the possible septic conditions within the sludge blanket and the production of a much thicker sludge by flotation, i.e. 4 to 5 percent solids vs. 1 to 2 percent solids. Flotation will require more skilled operations however.

The choice of coagulant involves lime,  $\text{Al}^{+3}$ ,  $\text{Fe}^{+2}$ , and  $\text{Fe}^{+3}$ . If aluminum or iron is chosen, the means of addition can either be by electro-chemical means or by addition of salts such as ferric sulfate, ferric chloride, ferrous sulfate or ferrous chloride. If the salt form is used, the ferric salts have an advantage over alum in that alum carries 14 waters of hydration; thus there is a low percentage of useful aluminum ion in a given weight of alum. Ferric chloride on the other hand, carries few waters of hydration; lower doses of it would be required than if alum, or even ferric sulfate, were used. Ferric chloride also dissolves rapidly.

Lime has frequently been the coagulant of choice for many waste treatment applications but its dosage is generally much higher than the dosage required for iron salts or aluminum salts, particularly if the water has a high alkalinity since lime reacts with the alkalinity. Also, there are indications that lime does not work well in flotation units unless the magnesium content of the water is high (Ref. 129).

Accordingly, if a coagulant is to be used in the salt form, ferric chloride is considered to be the best coagulant because it is effective, it results in the lowest shipping weight requirements and it is easily dissolved.

The alternative procedure for dosing water with a coagulant is to use electrochemical means of addition. General Electric Company (Ref. 121), in a study of treatment of wastes with approximately the same characteristics as those anticipated at a bare base, used an electrochemical means of adding coagulant in the form of  $\text{Fe}^{+2}$ . Solid iron was used as the anode and the current supplied stoichiometrically converted Fe into  $\text{Fe}^{+2}$ . It was observed, however, that for 75 mg/l of  $\text{Fe}^{+2}$  added electrochemically, it was necessary to add 50 to 100 mg/l of sodium aluminate to assist in coagulation. Problems were also encountered with electrode clogging. Further research may show the addition of coagulant such as sodium aluminate to be unnecessary, however.

On the basis of analysis of available data, it is recommended that ferric chloride be used as a coagulant and that it be added in the salt form. Electrocoagulation should be closely studied on bare base-type wastewaters in comparison with ferric chloride, however. The obvious advantage if it could be used as a replacement would be that the total weight of chemical which would have to be supplied would be much less. The weight of solid iron would be approximately 1/3 the weight of ferric chloride assuming  $\text{Fe}^{+2}$  acts

similarly to  $\text{Fe}^{+3}$ . Instead of chemical storage and chemical feed it would be possible to use an electrical source and an electrode system which has the potential of being more simple in terms of operation.

Using ferric chloride as the chemical of choice and 300 mg/l as a required dose, 2610 lbs/1000-man increment are required per month. The cost would be \$264 plus shipping costs. Solid iron bars would probably be available at a much reduced cost.

A polymer may also be required to improve solids removal. Laboratory studies should be undertaken to determine if polymer is necessary and if so, to determine the optimum dose. Assuming a 1 mg/l dosage, 9 lbs/month at a cost of \$13/month is required for each 1000-man increment. In general, polymer is supplied in the solid form and stock solutions are prepared on site. A chemical feeder would be required to add this chemical to the waste flow. The cost of coagulant and polymer feeders and associated equipment are estimated at \$4000.

### (3) Filtration

Regardless of whether flotation or upflow clarification is used, a filtration operation must follow these processes for good suspended solids removal. Microscreening is a possible alternative but it is subject to severe operational problems, such as flux decline when variable solids loadings and flow rates are encountered. Accordingly, microscreening is not considered to be a serious candidate for inclusion in the base waste treatment system.

The alternatives for various types of media filtration include sand or multimedia filtration in either the gravity or pressure mode, or a special type of filter such as that produced by the Hydromation Filter Company of Livonia, Michigan, wherein radial flow through synthetic filtration media

is used. Sufficient data are not now available to determine if the Hydromat Filter has a significant advantage over multimedia filtration.

For small-scale applications such as the bare base operation, pressure filters will give more flexibility of operation than will gravity type filters because they can operate over a wider range of head loss.

A multimedia filter is considered more advantageous than a sand filter. Sand filters have operated satisfactorily in water treatment for years and have also done a satisfactory job in tertiary treatment in many applications to date. However, multimedia filtration has improved on this operation because it gives a better distribution of pore sizes. A higher collection efficiency in terms of pounds of solids per cubic foot of bed before a given head loss is reached is thereby made possible. Alternatives in multimedia filtration include using 2 layers of media, e.g. sand and coal, or 3 layers of media, e.g. sand, coal and garnet. It is felt that the additional layer of garnet does not add a significant operating advantage and does increase the complexity of the system somewhat. Accordingly, the sand-coal dual media filter is considered the best for the bare base system. The operation of this filter should of course be evaluated in pilot plant operation on bare base-type wastewater and compared, if possible, to the operation of the Hydromat filter.

A dual media pressure filter operated at a design overflow rate of  $5 \text{ gpm/ft}^2$  is used for the recommended system, subject to further research findings. The required internal diameter is 2.5 ft for a cylindrical, vertical filter. The media depth required would be in the range of 24 to 30 in. with approximately 1/2 anthracite coal and 1/2 sand. An additional 12 in. must provide for the underdrain system and 20 in. for media expansion during backwash. The total filter height would be approximately 5 ft. Filters

should be supplied with backwash and surface wash appurtenances. The cost of such a pressure filter and appurtenances is estimated at \$7500. It would also have a shipping volume of approximately 40 ft<sup>3</sup> and a shipping weight of approximately 2000 lbs/1000-man increment.

It is necessary to use an intermediate storage facility preceding the pressure filter in order to store flow when the pressure filter or adsorber is out of operation. A detention time of 45 minutes should be sufficient for this purpose; assuming a pumping rate of 1.75 times the average, a tank capacity of 1620 gal is required. This could again be of the 2 in. redwood variety. It would have a shipping volume of approximately 75 ft<sup>3</sup> and would weigh 1500 lbs/1000-man increment. The cost would be approximately \$600.

#### (4) Removal of Dissolved Organics

As shown in Fig. 7 the processes of adsorption or chemical oxidation can be used to remove dissolved organic substances to levels of 25 to 75 mg/l COD. Ozone is the most logical oxidant for chemical oxidation but unfortunately it is best suited only for polishing effluents in which only a small reduction in the concentrations of organic matter is necessary. A reduction from approximately 150 mg/l to 50 mg/l would be required in this instance, necessitating an ozone dosage of approximately 230 lbs/day. According to Cochrane Division, Crane Company, the capital cost of ozone equipment is approximately \$500 to \$1000 for each pound per day of ozone generating capacity. This would require \$115,000 to \$230,000 in capital costs for ozone equipment alone. Additionally, a very large contactor would be required to bring about this reduction because of the limited rate of ozone adsorption by the water and the slow rate of reaction. Thus, it can be seen that the cost of applying ozone is extremely prohibitive in this case.

Adsorption by activated carbon appears to be the best choice at the present time. Granular carbon has been evaluated extensively for treatment of wastewaters and has performed very satisfactorily. Granular carbon is considered to be a better choice than powdered carbon because lower dosages are possible, thereby necessitating a smaller supply of activated carbon, and because more efficient contact is possible for organic removal. Powdered carbon, if applied, would utilize either an upflow clarifier or separate rapid mix flocculation and sedimentation equipment and would undoubtedly require that the filtration step be subsequent to the adsorber for the removal of powdered carbon escaping the contactor. Also it is more difficult to utilize the countercurrent principle when powdered carbon is applied to wastewater.

The alternatives for applying carbon to sewage in the granular form include the two-stage packed bed columns or the moving bed adsorber. The two-stage columns have been shown to be more economical than either the one-stage packed bed or the three or four stage packed bed columns. If packed beds are used, it is also necessary to use backwash and possibly surface wash to provide for removal of any solids which may accumulate due to bacterial growths or solids which pass through the multimedia filter.

The moving bed adsorber involves only one stage. The flow is introduced at the bottom and clean water is removed at the top of the column; periodically spent carbon is removed from the bottom of the carbon column and fresh carbon is added at the top. It is felt that the moving bed adsorber lends itself to the countercurrent principle very readily and permits maximum utilization of carbon efficiency for adsorption. It also permits a minimum utilization of space, and it eliminates intercolumn

connections and rotation of the columns when saturated carbon is replaced by fresh carbon. This adsorber requires no extra volume for bed expansion during backwash since no backwashing is required. Also, there is no need for backwash facilities and surface wash appurtenances, pumps, etc. The use of a moving bed adsorber, therefore, is recommended for the bare base treatment system.

The fact that only one carbon column is used necessitates the use of a rather tall column if sufficient detention time is to be maintained, and if overflow rates between 4 and 10 gpm/ft<sup>2</sup> are to be used. The use of a flow rate of 4 gpm/ft<sup>2</sup> and an empty bed contact time of 40 minutes results in a required carbon depth of 21.4 ft. This height is too great for bare base use in view of the air transportability constraint unless the column is manufactured in two pieces and assembled on the field. It was decided to use 1.5 gpm/ft<sup>2</sup> instead, resulting in a shorter, stouter column 12.5 ft tall and 4.5 ft in diameter. The reduced flow rate may alter the kinetics of adsorption; thus the effect of the reduced flow rate should be examined in laboratory studies.

A sketch of the moving bed adsorber is shown in Fig. 9. This adsorber uses cones sloped at 45° to the vertical at top and bottom. The required carbon depth within the adsorber is 8 ft, the total column height is 12.5 ft. The tank is to be filled with 8 x 30-mesh granular activated carbon from top to bottom.

Assuming a carbon capacity of 0.5 lbs COD/lb of carbon, removal of all the suspended solids and 50 percent of the COD influent to the plant in the chemical clarification and filtration steps, and removal of 75 percent of the COD influent to the adsorber by the carbon, a carbon requirement of 5,320 lbs/month/1000-men can be calculated. At a cost of 35¢/lb,

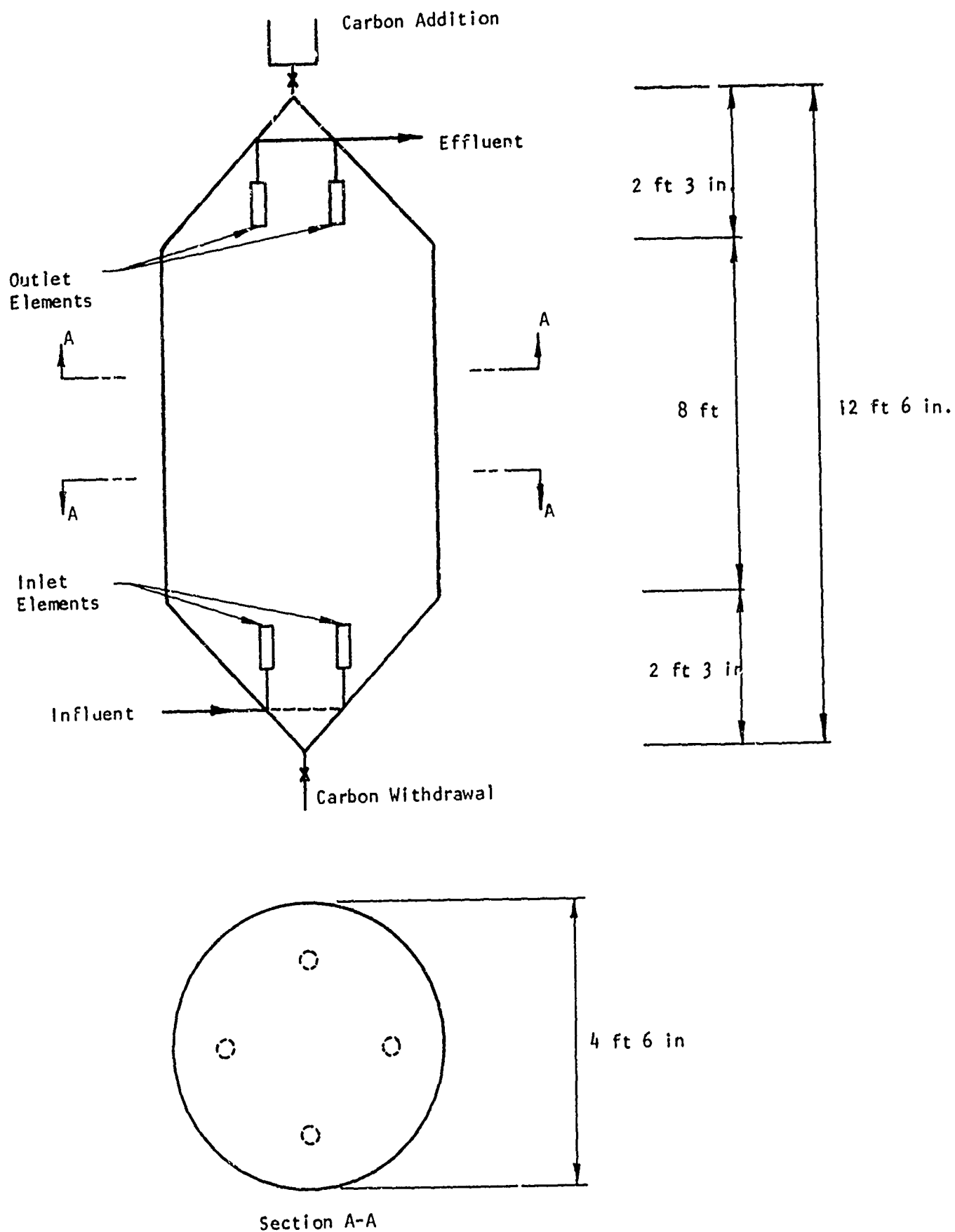


Figure 9. Moving-Bed Carbon Column Contactor



the cost on a monthly basis would be \$1860/month. The cost of the carbon adsorber is estimated at \$8000 and it has a shipping volume of approximately 200 ft<sup>3</sup>/1000-man increment and a shipping weight of 18,000 lbs when full of carbon.

Because of the requirement for large quantities of carbon per month, it is considered desirable to carry out studies to determine whether carbon can be replaced by a synthetic resin which can be regenerated within the adsorber column by caustic solution. Some preliminary research has been done on the use of resins for adsorption of organic matter but they are very limited and it is not known with any certainty whether or not this adsorbent will work for bare base type wastes. Accordingly, laboratory studies should be performed to determine whether or not synthetic resins are suitable replacements for activated carbon. A potential outcome of this study would be a significant reduction in the amount of material to be transported to the base.

Activated carbon packed beds have also been used for filtration of suspended solids in water treatment plants in the past. It is felt that the possibility of using a two-stage packed bed activated carbon system to replace both the moving bed adsorber and the dual media filter for suspended solids removal should be closely examined in the laboratory. This would have particular advantage in terms of conservation of space and simplicity of operation and, therefore, merits further consideration. Not enough is known about this process with respect to wastewater to say with any degree of certainty that it will operate satisfactorily, however.

#### (5) Disinfection

Disinfection may involve the use of halogens, chlorine, bromine, iodine or ozone as discussed earlier. Most experience has been obtained on

the use of chlorine for disinfection. The cost, availability and potential physiological effects appear to eliminate bromine and iodine from consideration (Ref. 122).

If chlorine is the disinfectant of choice, it should be used in the form of calcium hypochlorite instead of gaseous chlorine in order to eliminate the hazards posed by gaseous chlorine on a bare base. At a chlorine dose of 20 mg/l, 8.3 lbs of hypochlorite would be needed each day. This is preferably fed in solutions of about 2 percent available chlorine. A pump will be needed to convey the hypochlorite from the stock solution tank to the effluent from the carbon column. Such a system has the advantage of low cost and simplicity. The estimated cost of chemical feeder and related accessories is estimated at \$2000. Approximately 250 lbs/month per 1000-man increment of hypochlorite would be required. Calcium hypochlorite has about 70 percent available chlorine and costs about 38¢/lb; the cost per month then is approximately \$95.

Ozone should not be strongly considered for disinfection of the combined wastes. Because the COD is in the range of 25 to 75 mg/l, a dose of 15 mg/l is estimated as being necessary; this would require about 4 1/2 lbs/day of ozone. The capital cost of ozone generating equipment is \$2200 to \$4500. In addition, a significant operating cost due to a power requirement of 42 kw-hr would be entailed.

A finished water storage tank should be used to provide chlorine contact and to supply treated water for backwashing of the pressure filter. A 1750 gal tank should be used. This will provide 5 percent of the daily flow for backwashing and will also provide approximately 70 minutes of chlorine contact. The cost of the treated water tank, which is assumed to be 2 in. redwood, is approximately \$1000. It would have a shipping volume

of 75 ft<sup>3</sup> and a shipping weight of 1500 lbs/1000-man increment.

#### (6) Pumps

Several pumps are needed for the system shown in Fig. 8. Two pumps are needed to transport the flow from the equalization basin to the chemical clarification process. They should be identical so that rotation is possible; each pump should operate at a constant rate of 24 gpm. An estimated head loss through the rapid mix tank, flocculation tank, flotation unit and associated plumbing is estimated at 5 ft. Assuming the difference in elevation between the bottom of the equalization tank and the top of the rapid mix unit to be 15 ft, a pump capable of pumping at the rate of 24 gpm against 20 ft of head should be provided.

Two pumps should also be used to take the water from the intermediate collection tank following the flotation unit and pump it through the pressure filter and activated carbon adsorber into the treated water tank. Each should pump at a rate of 24 gpm and should be supplied in duplicate so that rotation is possible. Assuming the pressure loss is 15 ft through the multimedia filter, 10 ft through the carbon adsorber, 5 ft through the plumbing and an elevation difference of 10 ft, the total head requirement is 40 ft. Therefore, the pumps should each be capable of pumping at a rate of 24 gpm at a head of 40 ft.

A pump is also necessary to transport water from the treated water tank to the filter for backwash. Assuming the filter will be backwashed at a rate of 20 gpm/ft<sup>2</sup> including surface wash requirements, 98 gpm is the required flow. The total head requirement is estimated at approximately 20 ft.

The cost of the backwash pump is included in the cost of the filter. The cost of the other 4 pumps is estimated at a total of \$2000.

### (7) Miscellaneous

The cost of instrumentation for process control is estimated at \$5000.

The processes as discussed above have been considered separately. According to Crits (Ref. 108) an allowance of about 20 percent of the process cost should be included for marrying the equipment together. This yields a total miscellaneous cost of \$11,700.

### (8) Summary of Cost

The summary of the cost of chemicals for combined waste treatment is given in Table 17. Similarly, a summary of the capital cost estimates for the processes and process equipment are given in Table 18. Transportation costs are not included.

TABLE 17

COST OF CHEMICALS FOR TREATMENT OF THE COMBINED WASTES

| Chemical             | Cost (¢/lb) | Cost (¢/1000 gal) | Cost (\$/month) |
|----------------------|-------------|-------------------|-----------------|
| Carbon               | 35          | 175.0             | 1860            |
| Ferric Chloride      | 10          | 24.8              | 264             |
| Polymer              | 150         | 1.2               | 13              |
| Calcium Hypochlorite | 38          | 9.0               | 95              |
| Total                |             | 210.0             | \$2232/1000 men |

### c. Description of Waste Flow Through the Recommended Treatment Plant

A schematic of the recommended treatment system is shown in Fig. 8. Comminuted wastewaters flow into a 24-hour equalization tank. The contents

TABLE 18

## CAPITAL COST OF EQUIPMENT FOR TREATMENT OF THE COMBINED WASTES

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|   |        |
|---|--------|
| In-line comminutor                                  | \$ 500 |
| Equalization and blending tank (3 in. redwood)      | 4500   |
| Aeration and mixing equipment for equalization tank | 3500   |
| Chemical precipitation equipment                    | 20,000 |
| Filter (includes backwash pump)                     | 7500   |
| Carbon adsorber                                     | 8000   |
| Pumps P1, P2, P3, and P4                            | 2000   |
| Coagulant feeder and associated equipment           | 2000   |
| Polymer feeder and associated equipment             | 2000   |
| Hypochlorite feeder and associated equipment        | 2000   |
| Intermediate storage tank (2 in. redwood)           | 600    |
| Treated water storage tank (2 in. redwood)          | 1000   |
| Additional instrumentation                          | 5000   |
| Miscellaneous                                       | 11,700 |

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| Total Capital Cost/1000 men | \$ 70,300 |

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of this tank are kept fresh and well mixed by air-induced agitation. Some biological degradation of simple sugars, short-chain organic acids, etc. will take place in the aerated equalization tank under suitable conditions. The removal of these compounds at this early stage is a definite advantage since these compounds are most likely not adsorbed by activated carbon.

The equalization tank should be equipped with three level sensors capable of emitting signals that activate or deactivate certain pumps, as the situation may require. The positions of all three sensors are adjustable on site. When the fluid level in the tank rises to a predetermined maximum, the highest positioned level sensor automatically activates two 24 gpm pumps to pump the wastewater to the rapid mix unit. Each of these identical pumps is rated to pump wastewater at the average daily flow rate. When the wastewater level drops to a pre-set position, the intermediate sensor deactivates one of the two pumps so that only the average daily flow is pumped. At the lowest sensor position, all pumps are deactivated. A feature could be built into the system whereby rotation of the pumps is possible.

From the equalization and blending tank, the wastewaters are pumped into a rapid mix unit where ferric chloride and polymer are added. The flow is then directed to a flocculation unit where floc is allowed to develop. The wastewaters are then blended with the air-supersaturated return flow prior to entering the flotation basin. In the flotation basin most of the suspended solids, phosphorus and surfactants rise to the surface and are removed. The flotation basin is equipped with sludge removing equipment to remove any particles that settle.

The effluent from the flotation unit flows into an intermediate collection tank. This tank serves two purposes. It serves as a sump for the

two pressure filter influent pumps and it provides for surge storage when the pressure filter is being backwashed or the carbon filter is shut down for maintenance. The fluid level in this tank is kept below a certain level during normal operation to provide sufficient storage capacity.

A level sensor in the intermediate collection tank emits signals to the automatically adjustable flow regulator which is placed immediately after the pressure filter. As long as the water level does not exceed a preset position, the flow regulator permits flow at the average flow rate (24 gpm) through the filter. However, as the water level rises above this preset position, the adjustable flow regulator opens and allows more flow through the system. The higher the water level in the intermediate collection tank, the higher the flow rate allowed by the regulator; at a certain high level the flow regulator opens up completely and allows the maximum flow which the system will permit. This arrangement is designed to keep the water level in the intermediate storage tank at a preset elevation to provide for surge capacity for the effluent from the flotation unit.

The chemically clarified wastewaters are pumped from the intermediate collection tank into the dual media pressure filter which is equipped with an automatically adjustable flow regulator. When the pressure drop across the filter reaches 12 to 15 ft, the flow to the filter is automatically stopped and backwash is started. A backwash pump is used to convey treated water from the treated water storage tank and the backwash water is recirculated to the equalization tank.

The effluent from the pressure filter flows under pressure into the moving bed activated carbon contactor. The effluent from this contactor is chlorinated and directed to a detention tank to provide for sufficient chlorine contact time. This tank also serves the purpose of providing clean

water for backwashing the pressure filter.

No adjustment of pH is considered necessary for the chemical clarification step since ferric chloride is an effective coagulant over a wide pH range. Addition of ferric chloride lowers pH but this may be an advantage since lower pH values produce improved adsorption on activated carbon. The magnitude of the decrease in pH depends on the waste characteristics.

It is not possible at this stage to determine whether a pH adjustment step will be necessary for the treated water. If the treated water is to be discharged to a surface water body that does not possess enough buffering capacity, pH adjustment might be necessary to protect the ecology of that surface water. However, if the treated effluent is to be reclaimed by reverse osmosis, a low pH would be an advantage. For purposes of this report, it is assumed that no pH adjustment is required. If, however, pH adjustment is found to be necessary it can be easily achieved by adding base or acid to the effluent of the activated carbon column.

#### d. Sludge Treatment and Disposal

The various procedures for the treatment and disposal of sludges that are produced in the combined waste system are discussed in detail in Section X.

#### e. Effluent Quality from the Combined Waste Treatment System

Effluent quality from the treatment system can at best only be estimated at this stage. On the basis of a similar plant (Ref. 121), which includes electrocoagulation, upflow clarification, upflow expanded-bed carbon columns and chlorination. Influent  $BOD_5$  was 650 mg/l and total suspended solids were 600 mg/l. Chlorination and sludge incineration was also included in the system. A  $BOD_5$  of approximately 80 and suspended solids of 49 were



achieved by the system. Because of the inclusion of a dual media filter in the combined waste treatment system for the bare base, it is expected that the effluent suspended solids will be between 0 and 10 mg/l. Since much BOD is associated with suspended solids, it is also anticipated that the effluent BOD<sub>5</sub> and COD will be in the range 25 to 75 mg/l.

f. Alternative Treatment Systems

In the selection of the various processes for the recommended treatment system, several alternatives were considered. Although addition of ferric chloride was chosen as a means of coagulant condition, it is felt that electrocoagulation should be closely studied to determine whether or not it is a feasible alternative. Similarly, flotation was selected over upflow clarification for solids separation. If laboratory tests show flotation does not work well with bare base type wastes, upflow clarification should serve as its replacement.

If an effluent quality lower than anticipated for the proposed system is acceptable, it may be possible to bypass entirely the dual media filter and to use expanded bed upflow carbon columns instead which permit suspended solids to pass. This would increase suspended solids from approximately 0 to 10 mg/l up to the 50 mg/l range. The costs of the adsorber would not differ greatly but the cost of the dual media filter could be eliminated in this case. BOD and COD would approach 100 mg/l or more.

Activated carbon was selected as the adsorbent of choice but it is felt that synthetic resins hold much promise for replacing activated carbon and should be closely examined in laboratory studies to determine if this is true. The resin would probably cost 3 to 8 times that of activated carbon per cubic foot but the potential for eliminating the need to transport fresh activated carbon to replace saturated activated carbon needs to be closely examined.

Another alternative to the system outlined above involves batch treatment. The degrittied and comminuted wastewaters flow into one of two equalization tanks in parallel. Each tank would hold one day's flow; while one is being used for batch treatment, the other serves to receive and blend the wastewater flow. Coagulant, polymer, and powdered activated carbon are manually added in the desired proportions after a tank has been filled with waste. The rate of mixing in the equalization tank is controlled and varied so that rapid mixing, flocculation and sedimentation follow in sequence in the tank. The clarified wastes are then pumped at a controlled rate into a pressure multimedia filter, chlorinated, and directed to the treated water storage tank. The bottom of the equalization and blending tank would have to be shaped to provide for easy sludge withdrawal. Also a sludge pump and sludge line will have to be added to effect sludge removal at the end of batch treatment. A larger filter may also be required to account for the fact that flow is not continuous through the plant.

The above batch treatment system would make possible considerable savings in capital cost due to the elimination of the chemical precipitation equipment (rapid mix, flocculation, and dissolved air flotation units), the activated carbon contactor, the coagulant and polymer feeders, the intermediate surge tank and other accessories such as pumps and control valves. It would also eliminate mechanical difficulties associated with the recommended system. It would, however, require knowledgeable operation and greater quantities of chemicals, particularly activated carbon. Additionally, powdered activated carbon poses an explosive hazard if it is not handled properly. Also, it would be difficult to achieve the same effluent quality in this unit as compared to the recommended unit.

The use of powdered activated carbon in the batch treatment system, although less efficient with respect to removal of organic compounds than the countercurrent contacting procedure achieved by the use of column adsorbers, would affect a savings in cost when compared to the use of granular activated carbon. However, because of the higher dosages required, the explosion hazard, and the larger quantities of sludge produced owing to the presence of powdered carbon, it may be desirable to use granular activated carbon in its place. The granular carbon would be applied in a contactor similar to the moving bed adsorber in the recommended system.

Considerable laboratory testing would also be required before the batch treatment system could be designed and before the effluent quality of the system could be stated with any degree of certainty.

In the event that the start-up time criterion is lessened, activated sludge treatment of the waste may well become possible. If this were the case the report now under preparation by the authors of this report, to be submitted to the USAF in March 1972, should be examined to determine the best alternative.

## SECTION IX

### TREATMENT FOR WASTEWATER RECLAMATION

The subject of this section concerns the reclamation of effluent from the waste treatment system for the combined wastes. The RFP specifies a system which will produce a water quality which equals or exceeds the minimum requirements for drinking water as presented in the 1962 U. S. Public Health Service Drinking Water Standards. The reclaimed water could then be used as a source of supply for washracks, laundry, photo processing and shower and lavatory as well as other needs. There is some question as to whether or not the reclaimed water should be considered for general usage which would include human consumption. The American Water Works Association, in their policy statement on the use of reclaimed wastewater, has indicated the desirability of using reclaimed water for irrigation, industry and other non-consumptive uses, but does not feel the necessary technology exists at the present time to reliably produce water of a quality suitable for drinking (Ref. 87). The keyword in this statement is "reliably." The reliability of such a system remains to be demonstrated.

The system presented in this Section is not considered to be capable of reliably producing water of drinking water quality within the next year or two. However, it is a system which has the potential of being developed so that it can produce water suitable for consumption. The effluent definitely could be used for nonconsumptive needs at the present time.

In the event that a fresh water source is available at a bare base, the specific situation would have to be evaluated to determine whether it would be more advisable to reclaim effluent from the waste treatment plant, or to treat the fresh water to a level suitable for use. This question is

being dealt with at length in the research now under way on bare base water treatment systems and will be presented in report form in the Fall of 1972.

## 1. ALTERNATIVES FOR WASTEWATER RECLAMATION

The processes which have been considered for wastewater reclamation are given in Fig. 10 and include distillation, reverse osmosis and chemical oxidation - inorganic solids removal. Both inorganic solids and dissolved organic solids must be removed. If the recommended treatment system is used as outlined in Section VIII, the influent to the reclamation plant should have a  $BOD_5$  and COD in the range 25 to 75 mg/l and inorganic solids of approximately 1000 mg/l. Suspended solids will be in the range of 0 to 10 mg/l.

The 1962 Drinking Water Standards use carbon chloroform extract as a measure of the quality of water with respect to organic matter. Since carbon is used in the waste treatment system, the organics in the influent to the reclamation plant would be those poorly adsorbed on carbon and, therefore, not likely to be measured quantitatively in the Carbon Chloroform Extract test. The World Health Organization uses a COD of 10 mg/l as its standard for drinking water (Ref. 102) and this is considered more appropriate for reclaimed water.

### a. Distillation

Compared to sea water distillation, little research has been done on the distillation of wastewater. Some reports do exist which give insight into the process, however (Refs. 88, 89, 130). Studies at Oak Ridge National Laboratory (Ref. 88) indicate that both multistage flash distillation (MSF) and multieffect vertical tube evaporation (VTE) could be applicable to the purification of wastes which had been given tertiary treatment or its equivalent.

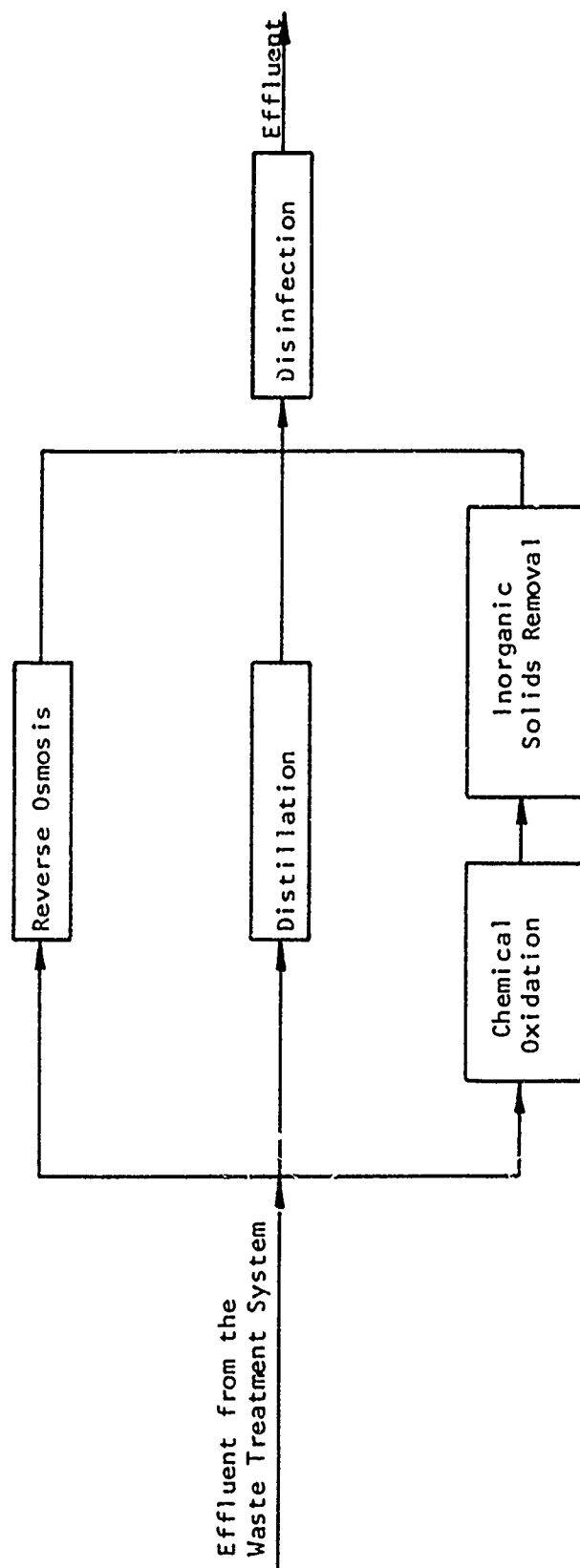


Figure 10. Alternatives for Wastewater Reclamation

Studies at the University of Florida (Ref. 130) showed that product water produced by evaporation of extended aeration, contact stabilization or high rate trickling filter effluent is not acceptable for reuse because of carryover of odor-causing organics in the temperature range 112 to 288°F. Treatment of the condensate with activated carbon can remove the odor, however. Ammonia also contaminated the product water; this could be removed from product water with ion exchange. Significant scaling problems also occurred with extended aeration and contact stabilization effluent. Corrosiveness was found by others to be a problem with the copper components of the distillation unit due to ammonia and sulfide attack (Ref. 88).

Inland distillation would require a heat sink to dispose of excess heat. If a natural disposal place is not available, a cooling tower, or similar apparatus, would be necessary.

Based on the distillation results discussed above, effluent from the waste treatment system for the combined wastes could be distilled readily. Scaling due to inorganics may be a problem. Since carbon treatment is part of the recommended waste treatment system, it is possible that volatile odorous compounds may not be a problem, but this remains to be evaluated. If effluent from the waste treatment plant is to be distilled, it may be possible to bypass the carbon column. The potential for organic scaling (Ref. 88) would have to be evaluated. In this case, odorous compounds would undoubtedly contaminate the product and necessitate its treatment with carbon. Control of chemical parameters such as pH would be necessary to minimize inorganic scale formation. A cooling tower may also be necessary.

As far as costs are concerned, a desk top study done for the Public Health Service in 1964 (Ref. 88) indicates that costs for the distillation of wastewater are expected to be 7 to 19 percent lower than those for the distillation of seawater. The costs for the latter using a fourteen-effect, long-tube, vertical, multiple effect evaporator were estimated at about \$1.15/1000 gal of product water for a 1 mgd plant. For smaller scale installations, the costs are expected to be considerably higher. Also, these costs do not include pretreatment of the wastewater for scale control, treatment of the distillate for removal of volatile organic components and disposal of excess heat.

Pilot plant studies have been used for development of techniques for evaporation of seawater. Distillation of wastewater will also require similar pilot studies to determine feasibility of the process and to develop design criteria.

#### b. Reverse Osmosis

The reverse osmosis separation process is a general and widely applicable technique in concept for the separation, concentration or fractionation of substances in fluid solution. It consists of pressurizing the solution to force the solvent to flow through a semipermeable membrane and withdrawing the membrane-permeate and concentrated solution.

The reverse osmosis membrane is the key component of the reverse osmosis system. These membranes are essentially barriers which prevent passage of molecules larger than certain minimum size. Molecules or ions smaller than a certain size pass through the membrane with the solvent. However, this ideal definition is not realized in practice. No precise molecular weight cut-off has been experienced with presently used membranes, and certain molecules tend to pass through the membrane more readily than others



regardless of molecular weight. The membrane which is presently dominating the reverse osmosis market is the cellulose acetate membrane. This membrane consists of a skin of dense cellulose acetate about 0.25 microns thick which surmounts a porous structure, about 99.75 microns thick (Ref. 90). The dense, noncompactable part of the membrane acts as the rejecting or semi-permeable layer, while the underlying substructure functions as a porous support. Other types of membranes have been studied and some are considered to have potential for use in place of the cellulose acetate membrane. Nylon is one such material.

The most significant application of the reverse osmosis process to date has been in the area of saline water conversion, but studies have also been carried out on wastewater purification and other applications. The reverse osmosis process has lately been receiving increasing attention for application to the advanced and tertiary treatment of wastewater. In spite of the rapid advances which are being made with respect to this process, it is still in the very early stages of development.

#### (1) Theory

The actual mechanism of transfer of water through a reverse osmosis membrane is not completely understood (Ref. 91). The most generally accepted theory is that water molecules pass through the membrane by successive transfer from one adsorption site to the next. In the case of cellulose acetate the adsorption forces arise from hydrogen bonding between the water molecule and membrane.

The rate of transfer of water through a membrane in reverse osmosis is directly proportional to the difference between the pressure drop across the membrane and the osmotic pressure, and inversely proportional to the thickness of the membrane (Ref. 92). This is shown in Eq. 1 where

$$J_1 = A (\Delta P - \Delta \pi) \quad (1)$$

where  $J_1$  is the solvent flux,  $A$  is the membrane constant which includes the membrane thickness,  $\Delta P$  is the pressure differential across the membrane and  $\Delta \pi$  is the osmotic pressure across the membrane.

The solute permeation rate is not a function of pressure differential but is a function of only the solute concentration differential across the membrane. This is shown in Eq. 2.

$$J_2 = B \Delta C \quad (2)$$

where  $J_2$  is the solute flux,  $B$  is the membrane constant, specific for a given membrane and the type of solute and  $\Delta C$  is the solute concentration differential across the membrane.

The concentration of constituents in the feed flow have a direct effect on product quality. The relation between concentration in feed and product flow is governed by the rejection ratios of the solutes, the type of membrane, the yield and other operational factors such as pressure and recycle ratio. The relationship between solute rejection ratio, yield and the permeate:feed concentration ratio as given by Ammerlaan (Ref. 103) is shown in Fig. 11.

Use can be made of Fig. 11 to determine permeate:feed concentration ratios of wastewater constituents for various yields, or percent recovery, of influent water. Such determinations assume a knowledge of the solute rejection ratio. It is apparent that percent yield becomes a trade-off with product quality for a given solute rejection ratio. As the yield is increased, product water quality decreases. If the types of solutes present are known together with their rejection ratios, product water quality can be predicted.

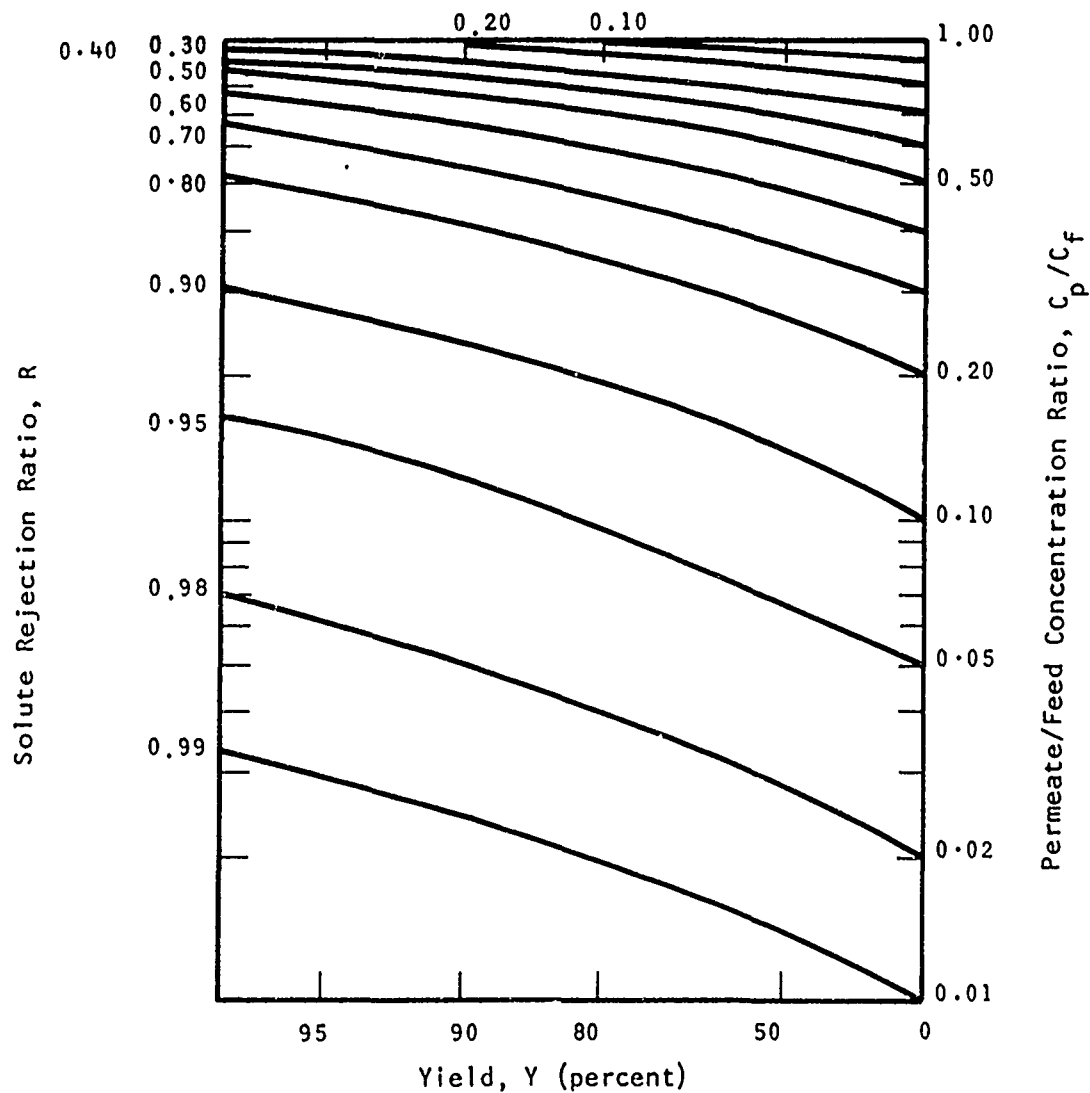


Figure 11. Permeate Concentration vs. Yield for Different Rejection Ratios

In the event that the types of solutes are not known, Fig. 11 can only be used to indicate the general effect of increased yield.

It is interesting to note that for a solute rejection ratio range of 0.5 to 0.7, the difference in permeate quality over the yield range of 80 to 95 percent is in the order of only 10 percent. However, the volume of brine requiring final disposal for a yield of 85 percent is 3 times that for a yield of 95 percent. In places where brine disposal poses a problem, this substantial reduction in brine volume at the expense of a slight decrease in effluent quality might be attractive. However, higher yields may also mean more operating problems because of salt precipitation.

## (2) Hardware

With the exception of the hollow fiber design concept, the membranes commercially available are basically the same. The difference in the various systems involves the manner in which the membrane is packaged for use. Golomb and Besik (Ref. 93) outlined the requirements for a successful osmotic module as follows:

- 1) It must provide proper mechanical support for the membrane.
- 2) It must provide for uniform distribution of the process solution over the entire membrane surface.
- 3) It must provide adequate hydrodynamic conditions for the process solution and permeate with minimum loss of energy.
- 4) It must have a high active membrane area/volume ratio.
- 5) It must be easy to dismantle and clean.
- 6) It must allow for easy membrane replacement and reassembly.
- 7) It must be highly reliable and operationally safe since it operates under pressure.
- 8) It must be as free as possible from leakages that could result from pressure-induced changes in dimensions.

9) It must be inexpensive to fabricate, repair and maintain.

Several types of osmotic modules have evolved that more or less meet the above criteria. Two different concepts have been employed in the design of these modules. The plate and frame, tubular and spiral wound concepts depend on relatively high permeation rates. The hollow fiber concepts rely more on very high surface area per unit volume with low permeation rates. Table 19 presents the membrane surface area to volume ratio of the various designs.

TABLE 19  
SURFACE AREA TO VOLUME RATIO FOR REVERSE OSMOSIS SYSTEMS

| Hardware                              | Membrane Surface Area to Volume Ratio<br>$\text{ft}^2/\text{ft}^3$ |
|---------------------------------------|--|
| Tubular (96)                          | 20   |
| Spiral Wound (96)                     | 250  |
| Plate and Frame (93)                  | 5 - 100  |
| Hollow Fiber (Nylon) (96)             | 5400   |
| Hollow Fiber (Cellulose Acetate) (96) | 2500   |

(a) Plate and Frame Units

This design provides a convenient solution to the pressure-containing problem. The system developed by Aerojet-General Corporation includes a membrane supported on a flat circular plate; several plates are contained in each pressure vessel. Product water emerges at the edge of plates in the smaller units, or in a central shaft in the larger ones. O-ring seals keep the feed and product liquid streams well

separated. Baffles located near the membrane surface induce turbulence in the feed stream (Ref. 90).

Though beset by a number of limitations, many installations of this type have been used for demineralization purposes. The disadvantages of this design include (Refs. 90,93), 1) high capital and operating costs, 2) problems with shortcircuiting and distribution of the feed, 3) narrow flow channels, 4) complexity of membrane handling and consequently higher probability of failure and 5) low membrane surface area per unit volume.

#### (b) Tubular Units

In this system the membrane is cast into a tube, generally 1/2 in. in diameter. The individual tubes are then packaged in modules of up to 36 tubes. There are a number of different tube arrangements. Havens Industries (now Calgon-Havens) casts their membranes inside a porous fiberglass tube which serves as both the membrane support and pressure vessel. The modules contain 18 such tubes. Aqua-Chem, on the other hand, casts their membranes on a porous cardboard type packing similar to a soda straw. This tube is then inserted into the module shell which is also the pressure vessel. Tubular membranes can be easily replaced in the field; however, with the Havens' design the fiberglass tube must also be replaced. With the Aqua-Chem unit, only the cardboard tube is replaced (Ref. 94).

The advantages of the tubular design are (Refs. 90,93), 1) it can handle solutions containing suspended solids and 2) the hydrodynamic conditions can be adjusted to minimize concentration polarization and fouling. This feature is to a large measure unique to tubular designs since it is extremely difficult to realize with other designs.

The disadvantages of tubular reverse osmosis designs include

(Refs. 90,93), 1) the large number of end fittings and connectors result in high capital and assembly costs, 2) a small volume to surface area ratio, 3) the necessity to enclose the tube exteriors to protect the purity of the permeate and 4) the expense of the support media.

#### (c) Spiral-Wound Units

This design, developed by Gulf General Atomic Corporation, consists of a sandwich arrangement composed of a porous backing material between two layers of membrane. The one end of the sandwich is attached to a perforated plastic pipe. The edges of the membrane are sealed; the sandwich is then wound around the perforated pipe using suitable mesh spacers. The membrane is then placed inside a cylindrical pressure container and the resulting module is connected in series with other such modules.

The main advantage of this design is that it realizes a high membrane surface area to unit volume ratio. However, it suffers from 1) severe fouling and clogging problems when handling suspended solids, 2) short feed flow paths, 3) high pressure losses and 4) difficulty in concentrate recirculation (Refs. 90,93). Also, difficulty has been experienced in developing reliable techniques for multi-leaf fabrication (Ref. 95).

#### (d) Hollow Fiber Units

Hollow fiber modules contain a very large number of hollow filaments, about 50 microns outside diameter, and 25 microns inside diameter. Space consuming and costly support media is not required. These fibers are bundled and their open ends are inserted into a plug of porous resin serving as a header. The bundle is inserted into a

cylindrical shell which serves as a pressure vessel. Pressurized liquid is pumped into the shell and permeate is collected from the ends of the hollow fiber bundle. The hollow fiber assemblies are characterized by a very high surface area to unit volume ratio. Two types of fibers are available; fibers made by Dow Chemical are spun from cellulose acetate while those made by Dupont are made from nylon and other polymers (Refs. 90,93). Although hollow fibers have a high surface to volume ratio, hydraulic inadequacies of the design may very well be a serious drawback (Ref. 96).

The disadvantages of the hollow fiber design include 1) the possibility of clogging by feeds containing suspended solids, 2) high head losses owing to the hydraulics of flow within the small diameter fibers and 3) end-sealing problems.

### (3) Operational Considerations

#### (a) Variation of Product Water Flux with Pressure

Flux is directly proportional to the difference between the applied pressure differential across the membrane and the osmotic pressure as given in Eq. 1. Fig. 12 is a plot of water permeation flux vs. effective differential pressure, covering present and projected technology for tubular, plate and frame, and spiral wound reverse osmosis units (Ref. 95). The actual flux for a given pressure, it should be noted, is a function of the type of membrane.

The values for water permeation flux shown in Fig. 12 are believed to be those for newly installed membranes. During prolonged operation, especially at high pressures, these membranes have a tendency to compact by plastic deformation with serious flux decline. The



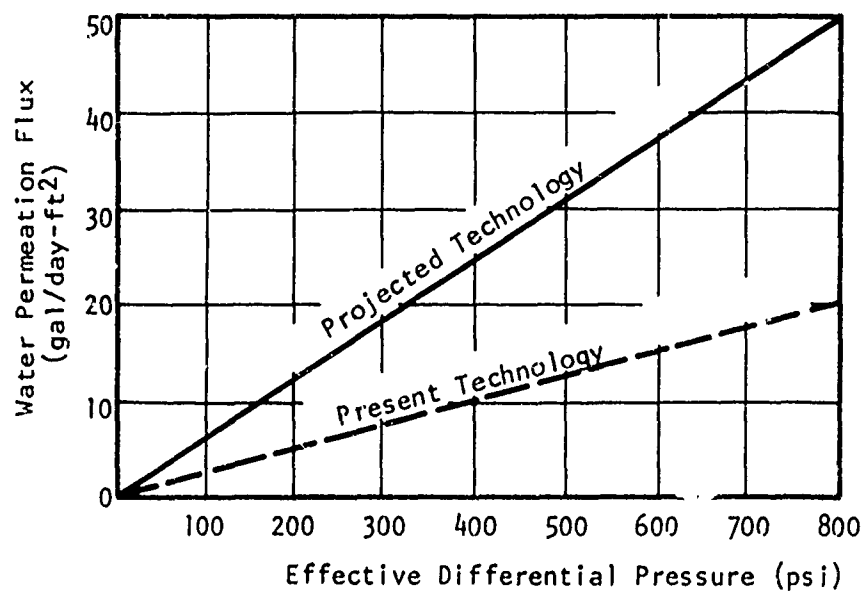


Figure 12. Water Permeation Flux vs. Effective Differential Pressure for Tubular, Plate and Frame and Spiral-Wound Reverse Osmosis Units

membrane also clogs with time causing a decline in flux. In trial runs performed by the Gulf General Atomic Corporation on spiral wound modules treating wastewater at the Pomona Water Renovation Plant of the Los Angeles County Sanitation District (Ref. 92), flux decrease ranged from 25 percent in 1500 hours for the tight membranes to 86 percent in 290 hours for the loose membranes.

(b) Yield

Yield, or water recovery, is related to the dissolved solids content of the feed water and the required permeate quality. The experience with the majority of reverse osmosis units operated on wastewater indicates water recoveries ranged from 75 to 95 percent. Average recovery for the Gulf General Atomic spiral wound unit operated at the water reclamation plant in Pomona was 75 percent, although it is reported that for a larger installation a more advantageous flow pattern would permit recoveries of up to 85 percent (Ref. 97). The Havens' tubular unit employed in Garrett Corporation's Water Purification Unit (Ref. 67) attained recoveries of 95 percent, but permeate water was of generally poor quality. Specific reasons for the poor quality are discussed under part (f), "Effluent Quality."

(c) Membrane Life

One of the most important factors determining the cost, and hence the feasibility, of reverse osmosis processes is membrane life. Membrane manufacturers have been projecting optimistic membrane life times of 2 to 3 years. Actual experiences with wastewater (Ref. 97), however, have not confirmed these projections. They indicate a more modest figure of 6 to 9 months. It is felt that further research will greatly improve

membrane life, however.

#### (d) Power Requirements

The electrical power consumption in reverse osmosis plants is a direct function of the feedwater pump motor mechanical efficiency, system operating pressure, energy recovery systems, plant capacity and recovery ratio. For design purposes, the power requirement can be based on 5 kw-hr/1000 gal (Ref. 98). For a plant treating 35,000 gal/day the power requirement amounts to an average 175 kw-hr/day.

#### (e) Concentrate Disposal

A byproduct of the reverse osmosis process is a concentrate requiring final disposal. In addition to having a high total dissolved solids content, it is certain that the concentrate produced by a reverse osmosis process on a bare base would also be characterized by a high organic content. Unless a large, saline body of water (sea or ocean) is nearby for final disposal, this concentrate might prove to be very expensive to dispose of. Depending on percent recovery, the reverse osmosis concentrate on a 1000 man bare base will vary from 1750 to 7000 gal/day (95-80 percent recovery). The forced evaporation and incineration of this concentrate would be expensive, requiring at least 1 gal fuel/4 gal concentrate. Lagoons utilizing natural evaporation could be used in arid regions.

#### (f) Effluent Quality

Most investigators report total dissolved solids reductions of over 90 percent and COD reductions of similar magnitude when product water is compared to the feed water. Reverse osmosis is even more selective for phosphate, but has only achieved about a 50-65 percent

reduction in nitrate nitrogen (Refs. 96, 97). A recent Advanced Waste Treatment Research Laboratory publication (Ref. 97) reports on typical removals that have been obtained in reverse osmosis units and these are listed in Table 20.

TABLE 20

TYPICAL REMOVALS FROM SECONDARY EFFLUENT BY REVERSE OSMOSIS\*

| Parameter  | Percent Removal | Parameter        | Percent Removal |
|------------|-----------------|------------------|-----------------|
| TOC        | 90              | Phosphate        | 94              |
| TDS        | 93              | Nitrate          | 65              |
| Turbidity  | 99+             | Ammonia          | 85              |
| Alkalinity | 90              | Organic Nitrogen | 86              |
| Chloride   | 80-85           |                  |                 |

\* These data were obtained using a cellulose acetate membrane, 450 psi pressure, at a flux of 8 gpd/ft<sup>2</sup>.

A recent study producing results not in conformity with the "typical" results shown in Table 20 are those obtained by testing the Water Purification Unit of the Aircsearch Division of Garrett Corporation built for the Mobile Army Surgical Hospital (MASH) and presented in Table 21 (Ref. 67). The Havens 6A tubular modules used during the test were new and had accumulated only a few hours running time during the manufacturer's quality assurance testing. They were, at the time of testing, the membranes with the tightest microstructure produced by the manufacturer. Synthetic hospital wastes were chemically precipitated and filtered before

TABLE 21  
REVERSE OSMOSIS PERFORMANCE IN THE  
MASH WATER PURIFICATION UNIT

| Determination          | Feed* | Brine* | Product* | Percent Rejection |
|------------------------|-------|--------|----------|-------------------|
| Aluminum               | 20    | 163    | 0.11     | 99+               |
| Arsenic                | 0.002 | 0.002  | 0.002    | 0                 |
| Barium                 | 0.05  | -      | 0.07     | concentrated      |
| Cadmium                | 0.02  | 0,120  | 0.005    | 75                |
| Calcium                | 9.1   | 360    | 1.1      | 88                |
| Copper                 | 0.04  | 1.39   | 0.004    | 90                |
| Iron                   | 12.5  | -      | 0.04     | 99+               |
| Lead                   | 0.26  | 0.77   | 0.03     | 88                |
| Magnesium              | 9.5   | 74     | 0.5      | 95                |
| Potassium              | 36.9  | 84.5   | 5.5      | 85                |
| Silver                 | 1.30  | -      | 0.003    | 99+               |
| Sodium                 | 315   | 3800   | 56       | 81                |
| Zinc                   | 4.6   | 12.3   | 0.53     | 89                |
| Chloride               | 263   | 2710   | 93       | 65                |
| Phosphate              | 157   | 1040   | 1.2      | 99+               |
| Sulfate                | 495   | 3540   | 11.4     | 96                |
| TOC                    | 136   | 1080   | 90       | 34                |
| Total Dissolved Solids | 1440  | 15,100 | 320      | 77                |
| Total Nitrogen         | 42.5  | 260    | 24.6     | 42                |
| COD                    | 312   |        | 140      | 56                |

\*Values expressed in mg/l

being fed to the reverse osmosis unit. From Table 21 it can be seen that the percent rejection of the inorganic ions closely parallels the "typical" results shown in Table 20. However, the reduction in COD and TOC at 56 and 34 percent, respectively, is greatly different. These results are surprising and difficult to compromise with the removals appearing in the literature. However, several factors may contribute to these lower reductions: 1) the water recovery of the Airesearch System at 95 percent is greater than the recoveries of 50 to 80 percent in other investigations and 2) the organic molecules most likely to leak through a reverse osmosis membrane were to a large measure not removed in the chemical clarification and filtration steps preceding reverse osmosis. These simple organic compounds would have been almost completely removed in systems in which biological treatment precedes treatment by reverse osmosis.

Reverse osmosis membranes have no effect on some ions and actually concentrate others. It can be seen from Table 21 that the concentration of barium actually increased in the permeate, and that of arsenic remained the same. Thus, barium, arsenic and any other ions with potential toxic effects must be given special consideration to assure that their concentrations in the permeate are well below toxicity levels.

#### (g) Position of Reverse Osmosis in a Treatment Sequence

The position of the reverse osmosis unit with respect to the carbon columns was given a great deal of consideration. If reverse osmosis is to be used in conjunction with the recommended waste treatment

unit, it could precede or follow the activated carbon columns. If the carbon columns are placed ahead of the reverse osmosis unit, the quality of water entering the reverse osmosis unit will be better, thus making it possible to operate the reverse osmosis unit at a higher percent recovery and lessening operational problems owing to membrane fouling. In addition, the removal of organics from the feed by activated carbon adsorption was found to be helpful in reducing the rate of flux decline (Ref. 92). This results in longer membrane life and hence in lower operational costs. However, the adoption of this treatment sequence results in greater activated carbon utilization rates and, therefore, higher costs for the carbon adsorption step.

From the above, it appears as if the proper way of resolving the question about the position of the reverse osmosis unit in relation to the activated carbon columns can be made on purely economic considerations. However, it is the investigators' feeling that reverse osmosis in its present state of development will function better if it receives influent containing as little organic matter as possible. This, of course, implies positioning the carbon columns before the reverse osmosis unit. However, reverse osmosis, among all the advanced waste treatment processes, is the one possessing the greatest potential for technological development. Future breakthroughs in flux rate, membrane life, etc., could well warrant the placement of the reverse osmosis unit ahead of the activated carbon column.

#### (h) Operational Problems

Adverse operational conditions such as high temperature, extreme pH values and other characteristics of the feed may result in

much shorter membrane life. It is sometimes necessary to remove particulate matter from the influent to prevent erosion of the membrane, fouling of its surface or blockage of thin passageways in some of the available systems (Ref. 95). Membranes should also be guarded from hydrolysis which serves to degrade the membrane. Hydrolysis is controlled in service by maintaining the pH of the feed in the range of 3 to 8, with an optimum value of between 4.5 and 5.5 (Refs. 92,94), and by guarding against operation at temperatures exceeding 125°F (Ref. 94).

Another phenomenon that can cause problems and shorten membrane life if allowed to go unchecked is that of concentration polarization. There is a tendency towards a buildup of a concentrated layer of solution immediately adjacent to the membrane surface for membranes with a high intrinsic flux rate for permeating waters (Ref. 95). This gives rise to two problems, the first of which is the fact that the effective osmotic pressure has to be overcome by applying higher pressure. The second problem stems from the fact that as the concentration of the solutes in the feed water adjacent to the membrane surface area increases, the solubility limits of some of the salts are exceeded. The precipitation of these salts on the membrane surface causes membrane fouling resulting in flux decline and shorter membrane life. To minimize the effects of concentration polarization, turbulence on the feed side of the membrane is required. High flushing velocities are maintained by recirculation, tapered flow, or otherwise. In some equipment, laminar flow in thin channels is used to eliminate this effect. Other measures include the addition of sequestering agents such as sodium tripolyphosphate to the feed to prevent salt deposition, and the adoption of a cleaning procedure to use once the membrane has become fouled. A promising method



for the latter is periodic rinsing of the membrane surface with an enzyme solution. Interestingly, the most effective enzyme solutions to date have been common commercial detergent pre-soak mixtures such as Biz (Ref. 96).

Biological attack of the membranes can also pose serious problems (Ref. 92). Good cleaning procedures are also required to prevent biological growth.

#### c. Chemical Oxidation - Inorganic Removal

In order to accomplish removal of organic compounds to approximately 10 mg/l as COD and total inorganic solids to less than 500 mg/l, chemical oxidation by ozone and a process such as ion exchange, electro-dialysis, or reverse osmosis, can be used.

Ozonation was discussed in Section VIII in connection with dissolved organic removal in the waste treatment system for the combined wastes. It was eliminated from consideration in that application because of its inability to accomplish economical removal of large concentrations of organics. It merits further examination in the reclamation system, however, since the quantity of organics requiring removal is much less. The quality of effluent from the waste treatment system has an expected COD of 25 to 75 mg/l. Thus 15 to 65 mg/l COD must be removed. Since each mg/l COD requires 2 mg/l ozone for removal and since approximately one-half of the ozone dosage is absorbed by the water (Ref. 123), 17.5 to 76 lbs/day of ozone generating capacity are necessary. The energy required for this generating capacity would be 150 to 600 kw-hr. According to Cochrane Division of Crane Company, the capital cost of generating equipment is estimated at \$38,000 to \$76,000 assuming removal of 65 mg/l COD. The reactor required for ozone contact and reaction would require

further study to design. Studies (Ref. 126) have indicated that staged cocurrent contacting is superior to other methods. These same studies showed a treatment cost of 7.7¢/1000 gal for a 10 mgd plant for a COD reduction of 20 mg/l.

The inorganic ion concentration requires reduction from approximately 1000 mg/l to 500 mg/l in order to be in accordance with the USPHS Drinking Water Standards. A paper by Dryden (Ref. 131) discusses the results of water reclamation studies at Pomona, California where reverse osmosis, electrodialysis and ion exchange were evaluated for their ability to remove dissolved inorganic compounds from carbon-treated secondary effluent. All processes could achieve the desired removal but ion exchange was found to be the least costly. Costs were 15, 23 and 25¢/1000 gal for ion exchange, electrodialysis and reverse osmosis, respectively. Product water had a total dissolved solids of 500 mg/l and the costs included 9¢/1000 gal for carbon pretreatment. Extrapolation of the costs to 0.5 mgd for ion exchange gave a cost of nearly 70¢/1000 gal. Brine disposal was not included in the costs.

#### d. Disinfection

Disinfection is an essential process prior to actual reuse. This process was discussed in some detail in Section VIII and many of the same considerations apply in reclamation as well. Chlorination by addition of calcium hypochlorite was selected as the recommended process in the waste treatment system over ozonation because of the potentially large ozone demand and the simplicity of chlorine feed. Bromine and iodine were considered less suitable than chlorine and ozone because of cost, limited availability and uncertain physiological effects (Ref. 122).

The best alternatives for disinfection in the reclamation system appear to be chlorination to a free residual of 1 mg/l after 30 min contact or ozonation to 0.4 mg/l after 10 min followed by chlorination. Each process will produce a water essentially free of virus and pathogenic bacteria, particularly since virus and bacteria are not small enough to penetrate the reverse osmosis membrane. Chlorination should follow ozonation because of the need to supply a residual in the distribution system to protect against contamination after the water leaves the plant. Even if the reclaimed water is not to be used for consumption, a chlorine residual should be maintained to protect against accidental consumption of contaminated water. Ozonation would have the additional benefit of oxidizing organic compounds which escape distillation or reverse osmosis, if either of these processes were to precede the disinfection step. Longer detention times and higher ozone dosages will be required if organic compounds are to be oxidized, however, in addition to disinfection.

In the event that water is to be reclaimed, disinfection of the waste treatment system effluent could probably be eliminated. The one exception would be if chlorination is required to prevent biological slimes from developing on the reverse osmosis membrane. Such slimes will not develop if the pH of the feed is below 6, however, regardless of whether chlorination were practiced or not (Ref. 125).

Chlorination has a distinct advantage over ozonation-chlorination in simplicity of operation, capital cost and operating cost. It will require larger quantities of  $\text{Ca}(\text{OCl})_2$ , however. The necessity of using ozone for oxidation of organics depends upon the quality of water produced by the preceding process and the desired effluent quality. If an ozonation process is made part of the reclamation system (e.g. reverse osmosis

followed by ozonation) for the destruction of residual organics, minor chlorination of the effluent is needed to maintain a residual disinfection capability in the distribution system.

## 2. SELECTION OF THE RECOMMENDED TREATMENT PROCEDURE

Reverse osmosis is considered to be the best alternative for the central component of the reclamation system. It can accomplish the necessary reduction in both COD and inorganic solids in one step and therefore has a distinct advantage in simplicity of operation over the chemical oxidation-ion exchange process. Reverse osmosis also has a lower energy requirement, i.e. an estimated 180 kw-hr vs. 150 to 600 kw-hr.

An analysis of distillation was hampered by a lack of information on the necessary degree of pretreatment, the quantity of blowdown and the quality of product. Pretreatment in addition to that given in the waste treatment system would probably be necessary to prevent inorganic scale formation on heat exchange surfaces. Further study may show that the degree of pretreatment as given in the recommended waste treatment system may not be necessary prior to distillation, however. Further treatment of the distillate to remove volatile, odorous organics and ammonia would probably be necessary in either case.

The quantity of residue from reverse osmosis poses a troublesome disposal problem. It is estimated that 80 to 95 percent recovery is possible, leaving 5 to 20 percent of the flow requiring some means of disposal. The same is also true to a lesser extent, however, for the alternative processes of ion exchange and distillation. Brine disposal is discussed in Section X.

Chlorination should be used as the disinfection step with chlorine

being applied as  $\text{Ca}(\text{OCl})_2$ . This process is simpler, costs much less and is easier to operate than an ozonation-chlorination process. Its power demand is, likewise, much smaller. In the event that the reverse osmosis effluent is not as good as required with respect to organic matter, ozonation-chlorination should be used in place of chlorination to remove additional organics. The reclamation system is shown schematically in Fig. 13.

The effluent from the reclamation system is expected to be of very good quality. The USPHS Drinking Water Standards should be met in every respect. In addition, a COD of 10 mg/l or less and a total dissolved solids content of approximately 50 mg/l is expected based on other studies (Ref. 125).

The reclamation system for the MUST program did not achieve desired levels of organic removal in initial testing (Ref. 67), however, and the reasons for this merit further consideration. COD concentrations of 100 to 150 mg/l were found in the effluent from the reclamation plant which consisted of reverse osmosis and activated carbon adsorption. The poor quality effluent resulted from the absence of an equalization basin, a poorly functioning solids removal process and an underdesigned carbon adsorber. As a result the quality of effluent coming from the unit was not as good as it might have been. The waste treatment unit recommended for the Bare Base Program includes a 24-hour, aerated equalization basin which should make efficient solids removal by flotation operationally possible. A related aspect is the biological degradation of the short-chain organic acids in the aerated equalization basin which would occur in a 1 but the most severe climates and serve to improve effluent quality.

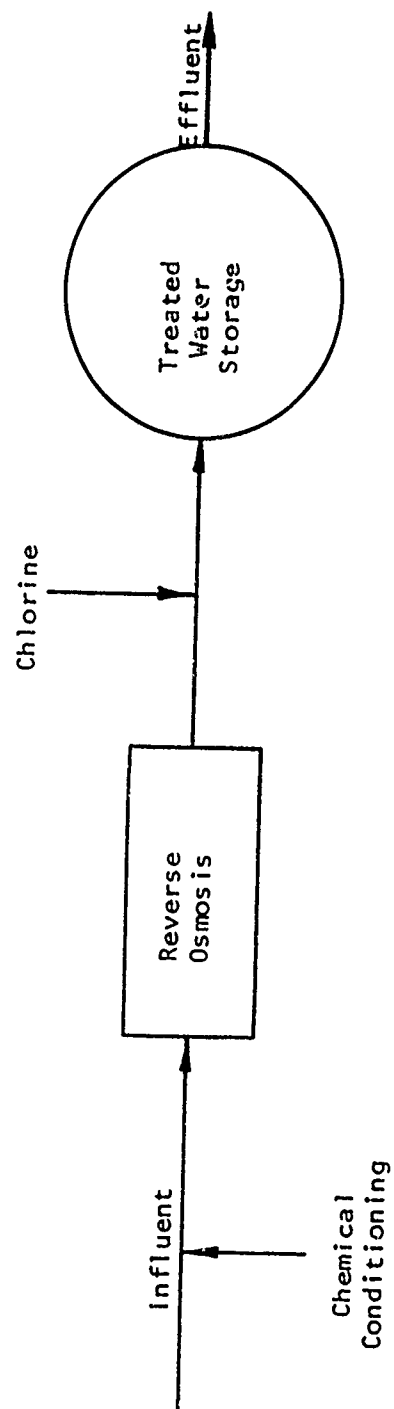


Figure 13. The Reclamation System

The waste treatment system recommended for the Bare Base Program also positions an activated carbon adsorber in advance of the reverse osmosis unit. This will improve the quality of water entering the unit and make possible the achievement of a better quality of effluent.

In the event that further research shows that a desirable COD cannot be reached by reverse osmosis when bare base wastewaters are treated, ozonation-chlorination should be used for organic removal and disinfection in place of chlorination for disinfection alone.

At present, reverse osmosis units using the tubular design appear to be more advantageous for bare base use than the other designs. However, reverse osmosis technology is fast moving and what holds at present might very well not hold in the near future. As such, a specific reverse osmosis design is not recommended for the post-1975 time period. Some assumptions were made to determine the approximate size, cost and power requirements for such a unit. Assuming the use of an Aqua-Chem tubular unit similar to the one recommended by Rex Chainbelt (Ref. 94) for U. S. Army usage on a similar waste, two units each with the following characteristics are required for 1000 men:

- 1) Dimensions: 36 modules in a unit 9 ft x 5 ft x 4 ft  
and the pump in a unit 3 ft x 2 ft x 2 ft
- 2) Weight: approximately 6000 lbs
- 3) Design Flux: 8 to 9 gpd/ft<sup>2</sup>
- 4) Power: 3.75 kw
- 5) Energy Requirement: 90 kw-hr/day
- 6) Cost: \$40,000 to \$50,000 per unit

Since two of the above units will be required for the renovation of the wastewater produced by 1000 men, the capital cost would amount to \$80,000 to \$100,000/1000-man increment. In addition, chemical feeding equipment will be necessary for pre-conditioning the influent to the reverse osmosis unit and also for chlorinating the effluent from this unit. The calcium hypochlorite feeding equipment described for the waste treatment unit can be used for chlorination. Higher capacity equipment will be required to treat both the effluent from the waste treatment unit and the reverse osmosis, however. The additional cost for providing this larger capacity is estimated at \$1000/1000-man increment. Equipment for feeding sodium tripolyphosphate to the influent to the reverse osmosis unit to prevent membrane fouling is estimated to cost \$2000/1000-man increment. Thus, the total capital cost for wastewater reclamation based on the use of reverse osmosis and chlorination as the principal process ranges from \$93,000 to \$103,000. This estimate is based on a design flux of 8 to 9 gpd/ft<sup>2</sup>. In the future, if membranes are developed that can reliably and consistently achieve flux values of 16 to 18 gpd/ft<sup>2</sup>, one of the above reverse osmosis units would suffice and the capital costs would be reduced to \$43,000 to \$53,000. The capital costs cited do not include costs for brine treatment and disposal.

Based on a dose of 5 mg/l of sodium tripolyphosphate to the influent to the reverse osmosis unit and a dose of 29 mg/l of calcium hypochlorite (20 mg/l of available chlorine) to the effluent from the reverse osmosis unit, the requirements and costs for chemicals are shown in Table 22.



TABLE 22  
CHEMICAL REQUIREMENTS AND COST FOR WASTEWATER RECLAMATION

| Chemicals                  | Requirements    |                                     | Costs            |               |                                   |
|----------------------------|-----------------|-------------------------------------|------------------|---------------|-----------------------------------|
|                            | lbs/1000<br>gal | lbs/month/<br>1000-man<br>increment | ¢/lb<br>chemical | ¢/1000<br>gal | \$/month<br>1000-man<br>increment |
| Calcium<br>Hypochlorite    | 0.238           | 250                                 | 38               | 9.0           | 25                                |
| Sodium<br>Tripolyphosphate | 0.042           | 44                                  | 8.5              | 0.4           | 4                                 |
| Total                      | 0.280           | 294                                 | 46.5             | 9.4           | 99                                |

### 3. ALTERNATIVE TREATMENT PROCEDURE

The alternative to reverse osmosis is considered to be distillation. Distillation was not selected as the recommended system because of high costs, uncertainty with regard to pretreatment requirements, and the probable need to remove volatile organic compounds from the distillate. In the event that a reverse osmosis system will not produce the required effluent quality, distillation should be closely studied at pilot plant scale to answer the above listed questions.

## SECTION X

### TREATMENT AND ULTIMATE DISPOSAL OF SLUDGE AND BRINES

A considerable number of unit processes are available for treatment and ultimate disposal and these can be categorized according to 1) in-plant treatment, and 2) ultimate disposal. In-plant treatment includes such processes as thickening, dewatering by vacuum filtration or centrifugation, digestion, drying and incineration. Ultimate disposal involves procedures such as lagooning, dumping at sea, land filling and land spreading.

The degree of in-plant treatment for a particular sludge is closely related to factors such as the cost of transportation, availability of a disposal site, and acceptability of the sludge in the environment concerned. Feasibility of one system over another, therefore, is difficult to evaluate precisely without understanding the exact environmental conditions imposed by the treatment site. From the standpoint of the Mobility Program, it is desirable to develop a system which is most flexible considering the unknown and uncontrollable environmental factors. This places much importance on in-plant treatment as opposed to other alternatives.

#### 1. SLUDGE CONDITIONING

Sludge conditioning refers to chemical or physical methods for improving the properties of sludge to permit water to be removed more rapidly when the sludge is subjected to thickening, dewatering or drying processes. Increased operating costs due to the added chemical costs are frequently compensated for by reduced physical size of sludge treatment units and associated labor.

#### a. Chemical Conditioning

Chemicals such as the ferric salts, lime, alum and polyelectrolytes are frequently used for conditioning of sludges. Chemical conditioning at the lowest possible cost is frequently achieved by a suitable combination of chemicals. Lime, for example, is advantageously used with other metal salts.

Use of polymers in sludge conditioning will result in the reduction of handling and storage of chemicals, a reduced quantity of sludge to be further processed and an increased fuel value per unit weight of the conditioned sludge. Typical doses are in the order of one percent of the weight of dry solids for polymers and 20 percent for inorganic chemicals.

#### b. Physical Conditioning

Various methods of physical conditioning for sludges are available. These include heat treatment, alternate freezing and thawing, use of inert solids, solvent extraction, application of electric potentials and ultrasonic vibration. None of these methods are in common use in the United States and, except for heat treatment, the technology of applying these methods is not well understood and commercial equipment is not readily available.

Commercial heat conditioning equipment includes the Porteous process, a modified Porteous process and the low pressure modification of the Zimpro wet-air oxidation process. Typical process conditions involve temperatures of 350 to 390°F at 180 to 210 psig for 30 min (Ref. 21). Advantages of heat treatment include elimination of odor and the concurrent destruction of pathogens. A disadvantage is the need for

recycle of a potent liquor, which is very difficult to treat properly, to the main treatment process.

## 2. SLUDGE THICKENING

Thickening is the process of removing water from sludge with the basic objective of reducing its volume. Such volume reduction generally leads to reduction of treatment costs due to savings in the physical size of sludge treatment system, labor and power.

The simplest method of thickening is gravity settling with or without the use of mechanical and/or chemical aids. Other techniques, such as air flotation and centrifugation, have been used for producing higher concentrations than are possible with simple gravity thickening.

### a Gravity thickening

A gravity thickener is a tank in which suspended solids are settled and concentrated by gravitational forces. Rakes and pickets are frequently provided to aid thickening and to collect the thickened sludge. The amount of solids which a gravity thickener can process differs appreciably with the type of sludge and the desired underflow solids concentration. Without knowing the exact settling properties of sludges, the following values are common' used for design purposes (Ref. 18):

| <u>Types of Sludge</u>       | <u>Solids Loading<br/>(lb/ft<sup>2</sup>/day)</u> |
|------------------------------|---|
| Primary Sludge               | 22  |
| Activated Sludge             | 4   |
| Primary and Activated Sludge | 8 - 12  |

The loading may vary a great deal for chemical clarification sludges depending on the type of chemical coagulant. The design procedures for a gravity thickener have been well described (Refs. 25, 26).

Unless there is a need to construct a separate thickener, a sedimentation tank can perform this function. If a separate thickener is required, flotation or centrifugation may be more promising.

b. Flotation Thickening

The basic concept of flotation thickening is the same as gravity thickening except that fine bubbles are introduced to increase the buoyancy of the sludge particles. The bubbles attach to the sludge particles and raise them to the surface where they can be collected.

The primary variables for flotation thickening are pressure, recycle ratio, feed solids detention period, air to solids ratio, type and quality of sludge and solids loading rates. These are all closely interrelated (Refs. 27, 28).

Representative data from field performance of dissolved air flotation equipment indicate that the following values are frequently used for design purposes:

- 1) Solids loading:  $0.5$  to  $5 \text{ lbs/ft}^2/\text{hr}$
- 2) Air requirements: max.  $0.03 \text{ ft}^3/\text{ft}^2/\text{min}$  or  $0.2 \text{ lb air/lb/solids}$
- 3) Recycle ratio: 40 percent

The principle advantage of flotation thickening as compared with gravity thickening is that it can thicken activated sludge or low specific gravity sludges of greases and oils more effectively. Also, a flotation thickener is smaller than the gravity thickener. Flotation thickening has the disadvantage that many parameters affect its performance. The unit is

not as easy to operate and the performance is not as consistent as the other thickening processes.

#### c. Centrifugation

Centrifuges can be used as thickeners by adjusting operational parameters. The large physical size of the units associated with gravity or flotation thickeners may render centrifugation more attractive. The fact that the centrifuge is highly compact may outweigh the disadvantages of high operation and maintenance costs and poor solids recovery.

### 3. DIGESTION

The two biological methods for stabilization of organic sludge are aerobic and anaerobic digestion. The purpose of digesting sludge is to reduce nuisances upon ultimate disposal. These processes are usually expensive, and have operational problems such as long start-up times and susceptibility to upset. Ultimate disposal is still required after digestion.

### 4. LAGOONING

Lagooning of sludge is by far the most simple process for sludge handling and disposal. Disadvantages associated with lagooning are that it requires large land area and it creates nuisance problems, particularly when undigested sludges are lagooned. The process is highly affected by climatic conditions.

### 5. SLUDGE DEWATERING

The primary purpose of sludge dewatering is to reduce the moisture content of sludge to a degree which allows ultimate disposal by

either incineration or by land filling or spreading. The process produces sludge solids concentration beyond that which thickening processes can accomplish. Typical values of the solids concentration of mechanically dewatered sludge range from 20 to 30 percent for organic sludges and 60 or more percent for some inorganic sludges (Ref. 21). Available dewatering equipment includes vacuum filters, centrifuges, mechanical pressure filters and sand bed filters.

a. Vacuum Filtration

Vacuum filtration is a major mechanical sludge dewatering process applicable to a variety of sludges. The drum of the unit is continuously passed through the sludge where it picks up solids to form a cake. A vacuum is used to draw water out of the sludge. The amount of solids which can be dewatered per unit time and unit area, and the dryness of the cake formed are highly dependent upon many variables (Refs. 29, 30, 31). The filter yield may vary from 2 to 10 lb/ft<sup>2</sup>/hr (Ref. 32). A wide variety of sludges can be easily dewatered as compared with centrifugation and the performance is not affected by varying environmental conditions such as temperature and humidity. Also, the dried cake can be incinerated. However, the unit is not easy to operate and the cost of operation can be high.

b. Centrifugation

Centrifugation is a solid-liquid separation process which uses gravity and centrifugal forces. Of the available types, the solid bowl centrifuge is generally considered to have the best combination of clarification and dewatering properties for use with most wastewater sludges. The cylindrical-conical solid bowl centrifuge has performed well in thickening operations (Ref. 21). Other types include basket and disc

centrifuges. In general, disc-type machines produce a poor quality cake even though they produce good clarification of the effluent liquid. The basket type, on the other hand, dewateres sludge effectively but the clarification function is poor (Ref. 33). Centrifuges can produce dewatered cakes generally comparable to those obtained from vacuum filtration but the solids recovery is generally poorer. The centrifuge is very simple and highly compact and requires only low or moderate maintenance. It is also very flexible in operation.

#### c. Sand Bed Drying

In spite of potential advantages associated with sand bed drying such as economics and simplicity, the applicability of the process to the system being sought does not appear to be promising. The process requires a large land area and construction materials may not be available near the construction site. The sludge should be stabilized before it is applied to the beds. Also, restoration of the site would be difficult and redeployment would probably not be feasible.

### 6. INCINERATION

Despite the economic disadvantage associated with incineration, burning of waste sludge has been gaining increasing attention as a potential alternative solution of sludge disposal. Incineration is the only practical means presently known that can accomplish maximum volume reduction and sterilization of the waste solids.

In addition to fuel and air requirements, factors such as retention time, temperature, and turbulence are important and should be considered for efficient burning. As in the case of any sludge treatment process, the



characteristics of sludge are the major factors governing the success of incineration. These include the moisture content, volatile and inert content, and the calorific value of the sludge. To be thermally self-sustaining, the moisture content of sludge is generally reduced by pretreatment.

Widely varying types of incinerators which are suitable for sludges are available. These include multiple hearth, fluidized bed, atomized spray, flash drying, rotary kiln, traveling grate and wet-oxidation incinerators. The principle differences between the types of incinerators are the type of heat exchange and the method of heat reuse to improve thermal efficiency.

In general, the advantages of incineration include 1) nearly complete combustion of organics, 2) large reduction of sludge volumes, 3) production of an inert ash that is easy to dispose of and 4) complete hygienic safety. However, the process is somewhat costly.

#### a. Multiple Hearth Incineration

Multiple hearth incinerators are common in large scale treatment. Although it has certain limitations in thermal efficiency, a multiple hearth incinerator can handle a variety of sludges including raw primary sludge, waste activated sludge, grease, screenings, skimmings and refuse (Ref. 34). A high solids content is essential for the process and thus it must frequently be preceded by a dewatering process. Multiple hearth incinerators are available with furnace sizes from 6 ft to 22 ft in diameter. As many as 12 hearths may be used. A conventional guideline for the design of such a unit is 7 to 8 lbs/hr of dried solids per ft<sup>2</sup> of hearth (Ref. 35). Available information indicates that the multiple

hearth incinerator is more applicable to large scale treatment systems than to small scale ones. Also, it does not appear applicable to dilute sludges and brines.

b. Fluidized Bed Incineration

A more recent type of sludge incinerator makes use of a fluidized reactor. The sludge particles are fed into a bed of fluidized sand supported by upward flow of heated air. The reactor serves as a large heat reservoir in which sludge is rapidly mixed with the media and efficient heat transfer is obtained. The solids are retained in the bed until the combustion reduces them to ash; the ash is carried out of the bed in the exhaust gases. The gases are treated by scrubbing to remove fly ash. The unit can easily be operated intermittently because of the thermal inertia of the sand bed.

Because of newness of the process, the performance of fluidized bed incinerators has not been well established. In general, the amount of excess air ranges from 10 to 20 percent and the temperature of the reactor is 1400 to 1500°F with a pressure of 2 psig (Refs. 17, 36). The fuel requirements for start-up and supplement can vary widely with the length of rest time and the nature of sludge to be incinerated (Ref. 37). The principal advantage of fluidized bed incinerators stems from the improved thermal efficiency and the maximization of reactor space utilization. The unit can burn varying types of sludge and extensive dewatering may not be necessary. It does have the distinct disadvantage of removing most of the ash with the stack gas, however. Much of the ash must be scrubbed out of the stack gas with a wet scrubber, thereby creating anew a water pollution problem.

#### c. Atomized Spray Incineration

Another recent type of incinerator involves atomization of ground sludge solids at the top of a reactor by means of a nozzle. The reactor is designed for high temperature and low pressure thermal destruction of solids. Preparation of solids for combustion involves grinding the solids, spraying the slurry into the top of a reactor to form an atomized suspension, drying and burning the sludge within the reactor, and collecting and separating the ash from the hot gases which are emitted (Ref. 38). This procedure should not be used on sludges which have a potential for clogging, however.

#### d. Rotary Kiln Incineration

This type of incinerator utilizes a slowly revolving kiln to provide mixing of the waste and to transport the waste through the incinerator (Ref. 132). A stationary combustion after-chamber is required to burn many of the gas stream components produced in the kiln. It is possible also to use the kiln for final stage burning after driving off volatiles and moisture in a conventional ignition chamber. A potential disadvantage is that some wastes tend to ball up and combust poorly in the kiln. Ash is periodically removed from the incinerator.

#### e. Wet Air Oxidation

Wet air oxidation is a different type of combustion than those described above. The process does not require preliminary dewatering. Combustion takes place in the absence of flame and in the presence of liquid water. This is accomplished by operating at high temperature and pressure conditions. A mixture of sludge and air is passed through heat exchangers and brought to the reaction temperature. As oxidation

takes place in the reactor the temperature increases. Oxidized products leaving the reactor are passed through heat exchangers to heat the entering sludge and air. Gases which are products of the decomposition are separated from the liquid and released through a pressure control valve to odor control equipment. The oxidized solids may be separated from the liquid by settling or other methods. For start-up and supplement, heat is supplied from an outside source such as a steam generator. For small installations the process can be operated on a batch basis. Pertinent data on the effects of temperature, pressure, the amount of air supplied and type of feed sludge on the performance of wet oxidation units are available (Refs. 39, 40).

#### e. Treatment of Sludge With Refuse

The treatment of sludges from wastewater with refuse in incinerators appears to be promising in that it can bring about a unified waste management system on a bare base. In addition to the nature and amount of sludges, the amount of refuse generated on the base must be considered. Dewatering of sludges may possibly be eliminated by blending the wastes from the two sources prior to incineration, use of heat from incinerated refuse for pre-drying of sludge, or use of heat from incinerated refuse for wet oxidation of sludge.

## 7. COMPARISON OF COSTS

The cost data for unit processes of sludge handling as they appeared in the literature are shown in Table 23. The wide fluctuation in costs for each process is affected by the difference in the plant size, characteristics of sludge, efficiency, and other physical and environmental

TABLE 23  
COST DATA FOR SLUDGE TREATMENT AND DISPOSAL \*

| Process              | Range of Total Costs<br>\$/Dry Ton |
|----------------------|------------------------------------|
| Gravity Thickening   | 1.5 - 5.0                          |
| Flotation Thickening | 6.0 - 15.0                         |
| Anaerobic Digestion  | 4.0 - 18.0                         |
| Centrifugation       | 5.0 - 35.0                         |
| Vacuum Filtration    | 8.0 - 50.0                         |
| Sand Bed Drying      | 3.0 - 20.0                         |
| Heat Drying          | 25.0 - 40.0                        |
| Incineration         | 8.0 - 40.0                         |
| Lagooning            | 1.0 - 5.0                          |
| Land Filling         | 1.0 - 5.0                          |
| Barging to Sea       | 4.0 - 25.0                         |

\* Data taken from Ref. 18

conditions imposed on each plant site. The data, it should be noted, are primarily applicable to municipal plants. The data do show relative costs associated with each process, however.

For the system which is being sought, sludge incineration is expected to result in the highest sludge disposal cost; however, it appears to be the most compatible with the concepts of the Mobility Program in that the end product is a residue which can readily be disposed of on land.

## 8. SLUDGES AND BRINES REQUIRING TREATMENT AND DISPOSAL

### a. Wastewater Treatment System

The major source of sludge on a bare base is the waste treatment unit which is described in detail in Section VIII. Given the waste flow of 35,000 gpd/1000 men a suspended solids concentration of the raw waste of 454 mg/l and assuming 90 percent removal of the suspended solids with a coagulant dosage of 300 mg/l  $\text{FeCl}_3$ , a solids production of 180 lbs/day on a dry weight basis can be calculated. Assuming 3 percent solids by weight in the sludge as it is removed from the flotation unit, 5900 lbs/day (710 gpd) of chemical sludge must be treated.

Without having an exact technique to predict the actual characteristics of the sludge, it has been assumed through this study that the sludge has properties intermediate to primary sludge and chemical sludge from tertiary treatment.

### b. Washrack Wastewater Pretreatment

Pretreatment of washrack wastes (see Section V ) for a 1000-man bare base results in approximately 250 lb/day (30 gpd) of oil and grease skimmings and grit which requires treatment and disposal.

c. Photographic Wastewater

The recommended procedure for treating photographic wastes is outlined in Section VI. This procedure results in 1330 lbs/day (160 gpd) of concentrated photographic wastes which require treatment and disposal. These wastes have a total dissolved solids concentration of about 40,000 mg/l.

d. Reverse Osmosis Concentrate

The brine which results from the application of reverse osmosis in the reclamation system depends on the percent recovery that is achieved. Assuming 95 percent recovery, approximately 14,600 lbs/day (1750 gpd) require treatment and disposal whereas 43,800 lbs/day (5250 gpd) result for 85 percent recovery. These brines will have a total dissolved solids content of 5000 to 20,000 mg/l depending upon the percent recovery.

9. SELECTION OF THE RECOMMENDED TREATMENT AND DISPOSAL PROCEDURE

a. Waste Treatment Unit Sludge

The procedure which appears most advantageous for treatment and disposal of sludge is centrifugation and incineration followed by land disposal of ash, either by burial or by spreading on land. Incineration is operable in all climates and in widely varying geographical locations. Use of incineration is also compatible with the site restorability requirement. The problem of air transportability of both hardware and fuel and the high capital cost appear to be the major disadvantages, however.

Of the various types of incinerators which were discussed in the initial portion of this Section, fluidized bed and rotary kiln incinerators appear best suited for bare base operation. Fluidized bed incineration has a major disadvantage in that a wet scrubber is necessary to remove ash

from the stack gas. Very large quantities of water are required to remove the ash. The majority of the ash produced in the incinerator leaves via the stack gas. Scrubber water must subsequently be treated for ash removal (Ref. 34). Also, some of the ash is likely to be soluble in the scrubber water and will be very difficult to remove. The rotary kiln, in spite of its more inefficient burning, does appear to be more suitable for bare base operation.

A rotary kiln incinerator has been developed by the Airesearch Division of Garrett Corporation which is compatible with the C-130 aircraft (Ref. 5). This incinerator has been tested in the U. S. Army's MUST Program and is undergoing modification at the present time. It has a burning rate of approximately 2000 lbs/day of mixed waste from a hospital complex, and could reasonably be expected to handle in the range of 35 to 45 gph of dilute waste (Ref. 64). It weighs 6000 lbs, has dimensions of 7 ft x 8 ft x 12 ft and has a 15 kw power requirement (Ref. 5).

It is not the intent of this report to recommend the use of a specific manufacturer's product. However, for purposes of this report, the Airesearch incinerator can serve as an indicator of what can be achieved with the Mobility Program constraints.

Assuming an incineration rate of 40 gph of dilute waste and a 10 hr operating day, two of these units would be required for incineration of the 710 gpd of sludge which will be generated by 1000 men in the waste treatment system. The fuel requirement for burning is undergoing study at the present time, but it is somewhere in the range of 1 gal fuel to 4 gal of dilute waste (Ref. 64). Assuming this value to be accurate, approximately 180 gpd of JP-4 fuel would be required to incinerate the waste.



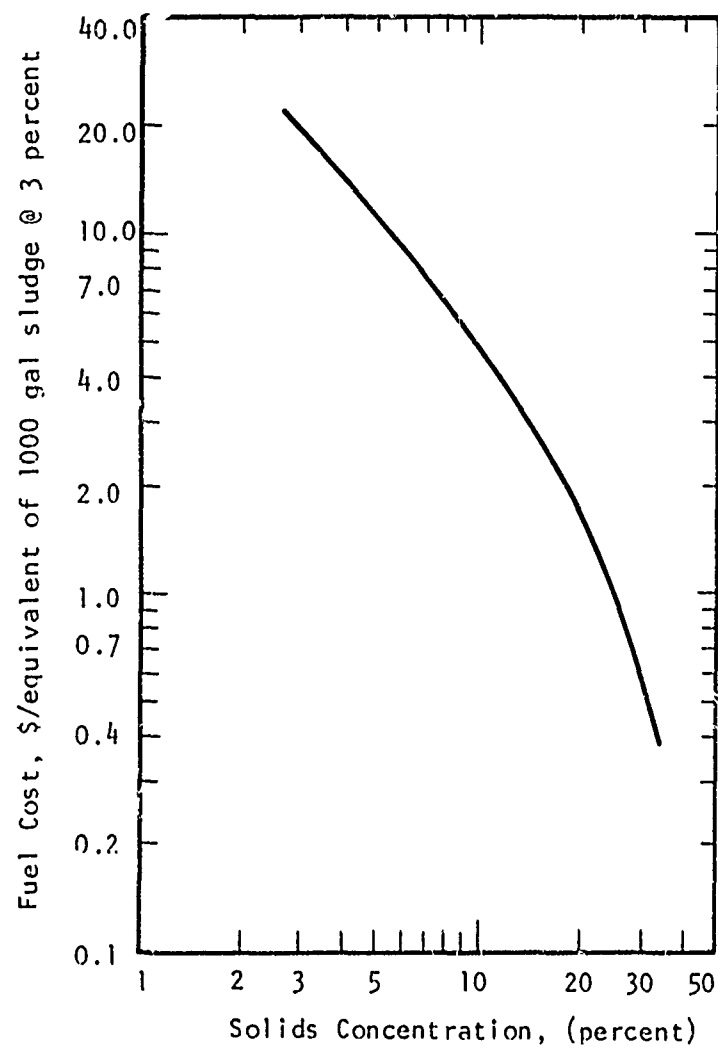


Figure 14. Effect of Dewatering on the Cost of Auxiliary Fuel in Fluidized Bed Incineration

The sludge volume can be reduced considerably by using a centrifuge to thicken the sludge prior to incinerating it. A cylindrical-conical centrifuge is considered best for this purpose (Ref. 21). Given that this centrifuge is used, a reasonable concentration for the sludge would be from 3 percent as the sludge is removed from the flotation unit to 25 percent as it leaves the centrifuge. An estimated solids recovery for this centrifugation would be 90 to 95 percent with polymer being used to increase the recovery of solids (Ref. 105). The feed rate for a centrifuge with a 9 in. diameter bowl is 2 to 6 gpm or 120 to 360 gph. Assuming a feed rate of 250 gph, the centrifuge is capable of treating 2500 gpd, assuming a 10-hour operation. This will provide capacity for up to 3000 men. The volume reduction accomplished by centrifugation is from 710 gpd to 90 gpd for a 1000 man base; the centrate is returned to the head of the waste treatment unit for treatment.

The new sludge volume of 90 gpd is well within the range of the capacity of one rotary kiln incinerator; the extra capacity may be used for other types of wastes. It should also be noted that fuel requirements are also significantly reduced because of the fact that the sludge approaches the point where it is thermally self-sustainable. A reasonable estimate, based upon consideration of different results reported in the literature, is that 10 gpd or 5 percent of the original fuel would be required. Fig. 14 shows the fuel cost for incineration as a function of sludge concentration. This graph pertains specifically to fluidized bed incinerators, but the same type of relationship is expected to hold for the rotary kiln and other types of incinerators.

Based upon this analysis, the incinerator should be preceded by centrifugation for solids concentration.

Because a commercially available incinerator meeting bare base criteria is not currently on the market, a good cost estimate for such a unit is not available. A figure of \$60,000 to \$120,000 is assumed, however, for the purposes of this report.

Estimated costs for centrifugation include 1) a capital cost of \$20,000, 2) a power requirement of 3 kw-hr/1000 gal of sludge and 3) a chemical cost of \$8.00/dry ton of solids for polymer to improve solids recovery.

#### b. Concentrated Photographic Waste

The concentrated photographic waste is one which has a very high potential for pollution and one which can be incinerated. Approximately 160 gpd require disposal for a 1000-man bare base. These wastes can be incinerated with the sludge from the waste treatment system in the rotary kiln incinerator. This waste has little heat value and therefore will require approximately 1 gal of fuel per 4 gal of waste, as previously assumed for the dilute aqueous sludge. Thus, 40 gpd of fuel will be required for burning this increment.

Potential problems involved in incinerating this waste involve heavy metal and particulate emissions (Ref. 109). Particulates can readily be removed by devices which can be attached to the stacks. If heavy metal emissions are a problem, it will probably be necessary to precipitate these prior to incineration. It is possible that the brine will evaporate better if it is mixed with sewage sludge prior to incineration.

#### c. Washrack Wastewater Skimmings and Grit

Approximately 30 gpd of oil, grease and grit come from the washracks. These can be incinerated together with the photographic waste

and waste treatment unit sludge in the rotary kiln incinerator.

In summary, a 1000-man bare base will produce a total of 280 gpd of waste, including centrifuged sludge from the waste treatment system, concentrated photographic waste and skimmings and grit from the washrack, which requires incineration. This is within the capability of one unit which can burn approximately 40 gph or 400 gpd on a 10-hour operating basis. Fuel requirements for this burning will be approximately 50 gpd.

The sludge treatment system is shown in Fig. 15. An estimated 160 lbs/day of ash will be generated by this procedure which can be disposed of in a land fill or simply spread on the land in some instances. If the ash contains soluble metal salts, care will have to be taken to landfill the ash such that it cannot be reached by water. Alternatively, the ash could be air transported to a more desirable disposal site.

#### d. Concentrate from the Reverse Osmosis Unit

The concentrate from the reverse osmosis unit is the most voluminous of the brines and sludges requiring disposal. Assuming 95 percent recovery, 1750 gpd must be disposed of. Ninety-five percent recovery, however, may possibly be unachievable in light of the quality requirements for the reclaimed water. It may well be that 80 to 85 percent recovery is the maximum possible. Given that 85 percent recovery is achieved, 5250 gpd must be disposed of. If recovery were 95 percent, four rotary kiln incinerators operating at the rate of slightly more than 400 gpd per 10-hour day, of dilute waste would be required for a 1000-man bare base for the reverse osmosis concentrate alone. Two incinerators would be required if 24-hour operation were possible. If the recovery were 85 percent instead, 12 incinerators would be required for 10-hour operation while 5 would be required for 24-hour operation. The fuel requirements

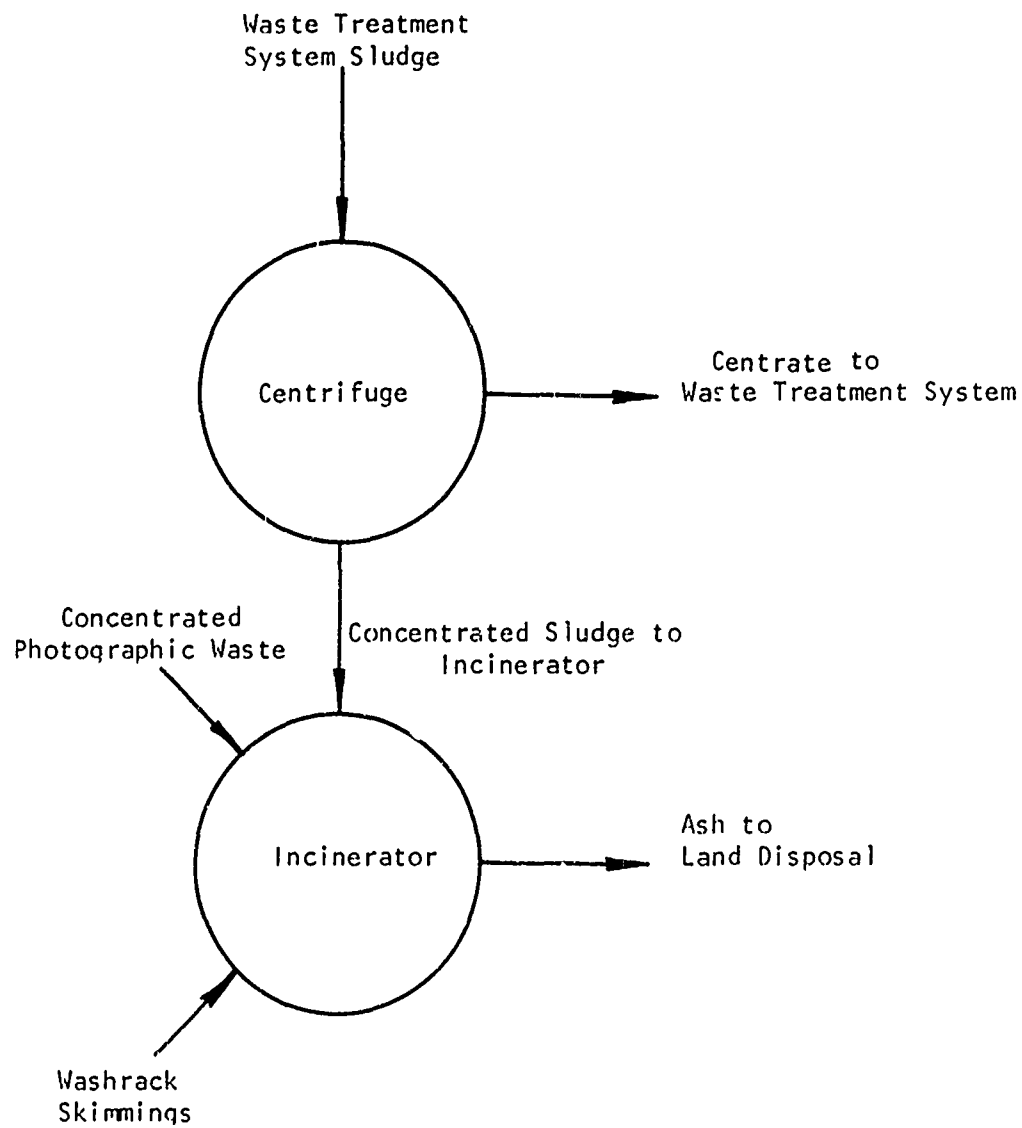


Figure 15. Sludge Treatment System

would range from 450 to 1350 gpd for the respective recoveries. It can be seen that incineration is out of the question because of the very large number of incinerators and the required quantity of fuel. Evaporators could be used to reduce the volume of hardware required and the fuel requirement, but the requirements would still be excessive.

One aspect of importance is the character of this brine. It should be remembered that the effluent from the waste treatment unit is suitable for discharge to the environment. The effluent has also been chlorinated such that there would be minimal danger resulting from pathogenic microorganisms. Based upon this consideration, it should be possible to dispose of this brine directly to the environment in most instances. This would particularly be the case where a body of water with sufficient dilution capacity were available, such as an ocean or an estuary. In the event that dilution capacity were not available, the possibility of either holding the concentrate in a small lagoon and 1) allowing it to evaporate in arid climates, or 2) discharging it when the necessary dilution capacity does become available due to precipitation, should be possible at the majority of bare base sites encountered.

If base engineers were to decide on the procedure at the bare base site, these men would have to be trained to make the necessary decision. In the event that climate and geographical considerations are such that discharge to the environment or lagooning were not feasible, the alternative of simply transporting this waste by aircraft or other means to a point where it could be disposed of should be considered.

It is also possible to evaporate the brine to significantly reduce volume prior to transport. Koenig (Ref. 133) reports costs of \$30 to \$40/1000 gal for capital plus operating costs for evaporation (1962 costs)

although information is not available on the size unit required. Control of emissions would be necessary since volatile organics exist in the brine. This procedure does have the advantage of being generally applicable, however.

In summary, it is recommended that the brine from the reverse osmosis unit be disposed of by a procedure to be selected by the base engineer for each site. It would be necessary to give him training to make this decision. In the majority of instances, this would include disposal to the environment, or holding in a lagoon for evaporation, or holding in a lagoon until such time as disposal to the environment is possible. This does violate the constraint of site restorability to a certain extent, but it is felt that it is warranted in this instance and that the effect of this violation should be minimal. In the event that none of these procedures are possible, the brine should simply be containerized and transported to a point where it could be discharged to the environment. It may prove desirable to evaporate prior to transport.

#### 10. ALTERNATIVE TREATMENT AND DISPOSAL PROCEDURE

The procedures which can be considered as alternatives to the above recommended procedure meet the bare base criteria in a less satisfactory manner. However, it may be desirable not to incinerate, at least in certain instances. In this event, centrifuged sludge from the waste treatment unit could be land filled or spread on the land. Depending upon the terrain and the location, it may also be possible to discharge the sludge to land without prior thickening.

It is felt that the most feasible way of disposing of the skimmings from the washrack units is incineration; there will undoubtedly

be incineration facilities available at a bare base site for handling at least this quantity of material.

In the event that disposal of reverse osmosis brine and concentrated wastes can not be dealt with on a case-by-case basis at the bare base site, the waste should be containerized and transported to a saline body of water for disposal. The feasibility of evaporating the brines prior to transport should be examined in this event.



## SECTION XI

### THE RECOMMENDED WASTE TREATMENT AND RECLAMATION SYSTEMS

#### 1. SUMMARY OF THE SYSTEM COMPONENTS

##### a. Aircraft and Vehicle Washrack Wastewaters

One tank will be located in the area of the vehicle washrack and one in the vicinity of the aircraft washrack for collection of the waste and removal of the grit and floating oil and grease from it. The wastes will be transported daily by truck to the waste treatment system for processing with the combined wastewaters. The floating oil and grease and grit are periodically incinerated.

##### b. Photographic Wastewater Treatment

The concentrated process wastes and the washwaters are segregated and collected separately. The concentrated process wastes are incinerated. The photographic washwaters, constituting 90 to 95 percent of the photographic wastewaters, are trucked to the waste treatment unit for processing with the combined wastes.

##### c. Human Waste Treatment

The human wastes include feces, urine, toilet paper and tank priming water. This waste is collected in tanks and burned in a batch incinerator. Seven human waste incinerators are required for the first 1000 men with each succeeding 1000 men requiring an additional four units.

##### d. Wastewater Treatment for the Combined Wastes

The combined wastes include the lavatory, shower, kitchen, dining room, laundry, hospital (excluding hospital human wastes), aircraft and vehicle washrack wastewaters and the photographic washwaters. The

latter three wastes are transported by truck to the treatment system. The combined wastes undergo treatment by the following processes:

- 1) Comminution
- 2) Aerated equalization and blending
- 3) Chemical addition and rapid mixing
- 4) Flocculation
- 5) Dissolved air flotation
- 6) Filtration
- 7) Activated carbon adsorption
- 8) Chlorination

e. Treatment for Wastewater Reclamation

Effluent from the waste treatment unit is conditioned with chemicals to prevent membrane fouling and is treated in a reverse osmosis unit. The product water is chlorinated and recycled for reuse.

f. Treatment and Ultimate Disposal of Process Sludge and Brines

The sludge from the waste treatment system is conditioned with chemicals, centrifuged and incinerated. The centr. e is returned to the equalization and blending tank.

The aircraft and vehicle washrack grit and skimmings are incinerated.

The concentrated photographic process wastes are incinerated.

The brine produced in the reverse osmosis process is disposed of in one of three ways: 1) in a body of water which provides adequate dilution, if available, 2) by evaporation ponds and lagoons in arid regions or 3) by transportation to a suitable disposal site if the first two alternatives are not available. The base engineer would be required to decide the best procedure and would require training for this.

## 2. SITING FOR A TYPICAL BARE BASE

The actual siting of the wastewater treatment and reclamation components can be made only after a knowledge of site topography. However, for illustration purposes, a siting of wastewater treatment and reclamation components is shown in Fig. 16. Favorable topography is assumed for the siting shown.

## 3. EASE OF START-UP AND RELOCATION

The recommended systems meet the Bare Base Mobility Program start-up and relocation requirements. The different units all can be designed to conform to the size and weight constraints imposed by the program. Detailed designs were not made as a part of this study, however. The majority of the components can be designed so that they need only be assembled with easy connect-disconnect type joints for operation. The tanks can be prefabricated and would have to be assembled and disassembled at the bare base site. However, this can be done very quickly and easily.

The system is ready for operation at full efficiency as soon as it is assembled. In view of the fact that the system is physico-chemical in nature, no start-up time is required for development of a biological mass. Therefore, it is the opinion of the investigators that the 72-hour start-up time constraint can be met easily.

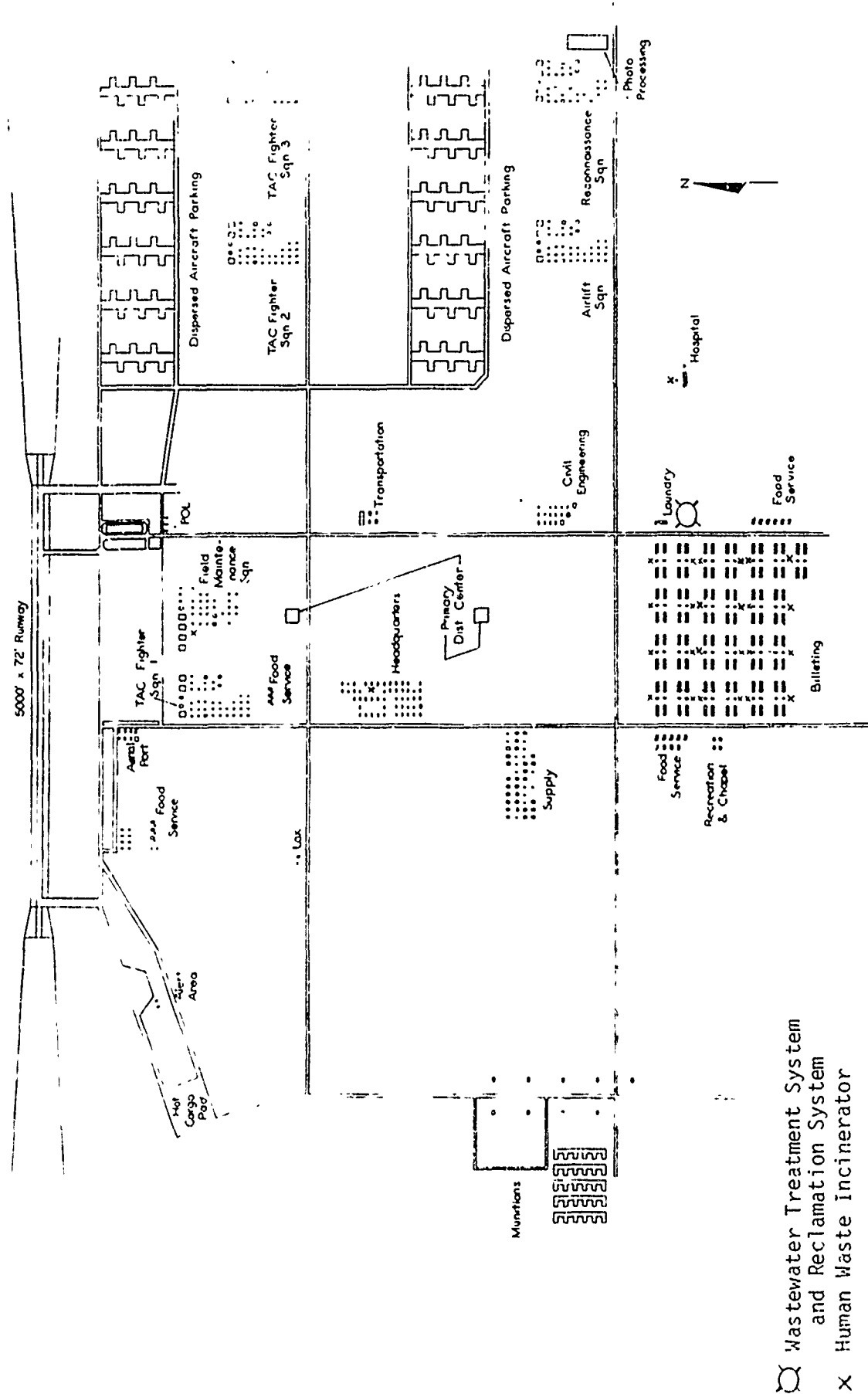


Figure 16. Siting for the Wastewater Treatment and Reclamation Systems

#### 4. POWER AND ENERGY REQUIREMENTS

##### a. Waste Treatment Unit

The maximum power requirement for the waste treatment unit is estimated at 11 kw/1000-man increment and the energy requirement is estimated at 250 kw-hr/1000-man increment/day.

##### b. Reclamation Unit

The power requirement for the reclamation unit is estimated at 7.5 kw/1000-man increment and the energy requirement is estimated at 180 kw-hr/1000-man increment/day.

##### c. Human Waste Unit

The maximum power requirements for the human waste incinerator are estimated at 2.45 kw for the first 1000 men and 1.4 kw for succeeding 1000-man increments. The energy requirement is estimated at 34 kw-hr/1000-man increment/day.

##### d. Sludge and Brine Handling and Disposal Unit

The power requirements for these units are estimated at 16 kw for the first 1000 men and 15 kw for succeeding 1000-man increments up to 3000 men. The power requirements for 4000 men are estimated at 62 kw and 15 kw for succeeding 1000-man increments up to 6000 men. The energy requirements are estimated at 150 kw-hr/1000-man increment/day.

#### 5. WASTEWATER COLLECTION SYSTEM

Transportation of the various wastes for combined treatment from the point sources to the waste treatment unit is by either truck or pipeline. The following wastewater sources are conveyed to the waste

treatment unit by pipeline: 1) lavatory and shower wastewaters from the billeting area, 2) kitchen wastewaters in the vicinity of the billeting area, 3) dining room wastewaters in the vicinity of the billeting area and 4) laundry wastewaters. This would mean a flow of 27,500 gpd/1000-man increment.

The topography of each particular base will determine the most advantageous location for siting the waste treatment unit and also the best method of sewerage to this unit. Based on locating the waste treatment unit as shown in Fig. 16 and assuming that the lavatory, shower, kitchen and dining room wastewaters can all be directed by gravity to a central sump from where they can be pumped under pressure to the waste treatment unit, and assuming the laundry wastewaters are pumped directly to the influent of the waste treatment unit, the required length of 2 in. pipe is 4200 ft/1000-man increment. Two in. piping is considered by the investigators to be the minimum desirable pipe for conveying the types of wastewater of concern. For weight considerations PVC pipe with quick connect-disconnect fittings and connectors may be used. Pumps would also need to be supplied in duplicate to convey the wastewaters from the sump and to convey the laundry wastewater to the waste treatment unit. Pump sizing again depends on topography, but assuming an elevation differential of 30 ft from the bottom of the sump to the top of the equalization and blending tank in the waste treatment unit, and 20 ft from the bottom of the tank collecting the laundry wastewater to the top of the equalization and blending tank, the capacity of each of the two pumps pumping from the sump would be 15 gpm at about 40 ft of head, and 4 gpm at about 30 ft of head for each of the two pumps conveying laundry wastewaters. A 3000 gal

sump tank would be required for the collection of the lavatory, shower, kitchen and dining room wastewaters, and a 1000 gal tank would be required for the collection of the laundry wastewaters.

The following wastewaters are transported by tanker truck to the waste treatment unit because of their distance from this unit:

- 1) Aircraft washrack wastewater
- 2) Vehicle washrack wastewater
- 3) Photographic washwater
- 4) Hospital wastewater (excluding the hospital human wastes)
- 5) Other minor nonhuman waste streams produced at different points in the base

For 1000 men, this flow would amount to about 6000 gpd.

A trailer-mounted tank with a 100 gpm pump should be provided to convey these wastewaters to the waste treatment unit. A 3000 gal tank should be provided at the aircraft washracks for collecting this wastewater, and 1500, 1000, and 1000 gal tanks should be provided for collecting the photographic, vehicle washrack and hospital wastewaters, respectively.

## 6. FREEZE PROTECTION

Where freeze protection is necessary, the different treatment units and collection tanks, with the exception of the equalization and blending tank, need to be housed. Since the wastes will possess heat and since the distances are short, it is felt that insulation of the piping will be sufficient to insure that wastewaters will not freeze during transport to the waste treatment unit. During periods of pump idle time, pressure conduits must be automatically back drained to the

sump to prevent freezing within the pipe. Housing of the treatment systems and collection tanks can be accomplished easily in collapsible structures similar to those used for other purposes on a bare base.

For the arrangement and siting assumed in Fig. 16, 4200 ft of 3/8 in. pipe insulation of approved quality will be required.

## 7. OPERATIONAL MANPOWER REQUIREMENTS

Assuming that a satisfactory design of the recommended system is achieved, an estimated 8 man hours/day will be required for operation of the 1000-man unit. The operator should be capable of running routine analytical tests for quality control and for diagnosing process failure. A maintenance crew should be available as needed for major equipment repair. An estimated 8 man hours/day will also be required to transport the washracks, photographic and hospital wastes to the treatment unit.

Economy of scale is expected as base size is increased. For example, it is anticipated that 16 man hours/day will be required for a 3000-man base and 32 man hours/day for a 6000-man base. The additional manpower requirement after the first 8 hours/day can also be of lower skill. An additional 4 man hours for each 1000-man increment above 1000 men will also be required for transportation of the waste.

## 8. VOLUME AND WEIGHT CONSIDERATIONS

All the components of the bare base wastewater treatment system can be made to conform to the requirements for the critical envelope of the C-130 aircraft and the 463-C loading system. The volume and weight considerations for the various components of the wastewater treatment system are found in Tables 24-28. It must be stressed that the



values given are only preliminary estimates and could differ from the actual values once the units are designed and fabricated. Also, certain units and accessories can be packaged together for shipping to save volume. For example, the flotation unit during shipping can well have various smaller units, pumps, valves and other accessories packaged in its hold to reduce the shipping volume requirements.

TABLE 24

## VOLUME AND WEIGHT CONSIDERATIONS FOR THE WASTE TREATMENT UNIT

| Description   | Shipping Volume<br>cu ft/1000-man increment | Shipping Weight<br>lbs/1000-man increment |
|---|---|---|
| Equalization and Blending<br>Tank (3 in. redwood)       | 800   | 16,000                                    |
| Flotation Unit and<br>Accessories                       | 450   | 6000                                      |
| Flocculation Basin<br>and Equipment                     | 100   | 1400                                      |
| Intermediate Collection<br>Tank (2 in. redwood)         | 75  | 1500                                      |
| Pressure Filter and<br>Accessories                      | 40  | 2000                                      |
| Carbon Column (full)<br>and Accessories                 | 200   | 18,000                                    |
| Treated Water Tank<br>(2 in. redwood)                   | 75  | 1500                                      |
| Other Components,<br>Accessories, Pumps, Fittings, Etc. | 200   | 10,000                                    |
| Total   | 1940  | 56,400                                    |

TABLE 25

## VOLUME AND WEIGHT CONSIDERATIONS FOR THE RECLAMATION UNIT

| Description                              | Shipping Volume<br>cu ft/1000-man increment | Shipping Weight<br>lbs/1000-man increment |
|--|---|---|
| Reverse Osmosis (2 units)                | 360   | 12,000                                    |
| Pump (2 units)                           | 24  | 500                                       |
| Feeders and Other Accessories            | 20  | 1000                                      |
| Chlorine Contact Tank<br>(2 in. redwood) | 50  |   |
| Total                                    | 454   | 13,500                                    |

TABLE 26

VOLUME AND WEIGHT CONSIDERATIONS FOR  
SLUDGE HANDLING AND DISPOSAL

| Item                        | Shipping Volume*<br>cu ft/first 1000 men | Shipping Weight*<br>lbs/first 1000 men |
|-----------------------------|--|--|
| Centrifuge and Accessories  | 100                                      | 5000                                   |
| Incinerator and Accessories | 800                                      | 6000                                   |
| Total                       | 900                                      | 11,000                                 |

\*The centrifuge is sized for handling the sludge produced by 3000 men. Therefore, this unit will not have to be duplicated unless the number of men exceeds 3000.

TABLE 27

VOLUME AND WEIGHT CONSIDERATIONS FOR  
HUMAN WASTE TREATMENT

| Description                                | Shipping Volume<br>cu ft | Shipping Weight<br>lbs |
|--|--------------------------|------------------------|
| Human Waste Incinerator                    | 37/unit                  | 450/unit               |
| Total/first 1000 men<br>(7 units)          | 259                      | 3150                   |
| Increment/additional 1000 men<br>(4 units) | 148                      | 1800                   |

TABLE 28  
VOLUME AND WEIGHT CONSIDERATIONS FOR  
WASTEWATER COLLECTION AND CONVEYANCE

| Item                                       | Shipping Volume<br>cu ft/1000-man increment | Shipping Weight<br>lbs/1000-man increment |
|--|---|---|
| 2 in. Pipe (PVC)                           | 190   | 2900                                      |
| Collection Tanks, Pumps<br>and Accessories | 350   | 7000                                      |
| Total                                      | 540   | 9900                                      |

#### 9. CHEMICAL REQUIREMENTS

##### a. Waste Treatment Unit

The chemical requirements for the waste treatment unit are shown in Table 29.

TABLE 29  
CHEMICAL REQUIREMENTS FOR THE WASTE TREATMENT UNIT

| Chemical             | Monthly Requirements<br>lbs/1000-man increment |
|----------------------|--|
| Activated Carbon     | 5320   |
| Ferric Chloride      | 2610   |
| Polymer              | 9  |
| Calcium Hypochlorite | 250  |
| Total                | 8189   |

b. Reclamation Unit

The chemical requirements for the reclamation unit are shown in Table 30.

TABLE 30

CHEMICAL REQUIREMENTS FOR THE RECLAMATION UNIT

| Chemical                | Monthly Requirements<br>lbs/1000-man increment |
|-------------------------|--|
| Calcium Hypochlorite    | 250  |
| Sodium Tripolyphosphate | 44   |
| Total                   | 294  |

c. Sludge and Brine Handling and Disposal

The chemical requirements for sludge and brine handling and disposal consist of very small quantities of polymer for conditioning purposes. An estimated maximum of 4.5lbs/day of polymer is required.

10. FUEL REQUIREMENTS

The fuel requirements include those for the incineration of the human wastes, the concentrated photographic process wastes, and the dewatered sludge. They do not include fuel requirements for the generation of power for plant operation. Table 31 summarizes the requirements for fuel/1000-man increment.

TABLE 31

## FUEL REQUIREMENTS FOR SLUDGE AND BRINE DISPOSAL

| Component   | Quantity of Fuel<br>gal/month/1000-man increment |
|---|--|
| Human Waste Incineration                                | 3750   |
| Sludge Incineration                                     | 300  |
| Concentrated Photographic Process<br>Waste Incineration | 1200   |
| Total   | 5250   |

## 11. COSTS

## a. Capital Costs

- (1) Waste treatment unit, \$70,300/1000-man increment
- (2) Reclamation unit, \$93,000 to \$103,000/1000-man increment
- (3) Human waste treatment, \$49,000 to \$147,000/first 1000 men;  
\$28,000 to \$84,000/succeeding 1000-man increment
- (4) Sludge and brine handling and disposal (does not include brine disposal of reverse osmosis concentrate), \$80,000 to \$140,000/  
first 1000 men; \$60,000 to \$120,000/succeeding 1000-man increment  
to 3000 men; \$260,000 to \$500,000/4000 men; \$60,000 to \$120,000/  
succeeding 1000 men to 6000 men
- (5) Collection system (includes pipes, tanks, tanker trailer and  
pumps), \$10,000/1000-man increment
- (6) Freeze protection (includes pipe insulation only), \$1500/1000-man  
increment

b. Fuel Costs

Assume 1 gal of fuel costs 10¢

\$525/1000-man increment per month

c. Chemical Costs

- (1) Waste treatment unit, \$2232/1000-man increment/month
- (2) Reclamation unit, \$99/1000-man increment/month
- (3) Sludge and brine handling and disposal, \$200/1000-man increment/month.

## SECTION XII

### RECOMMENDED TEST PROGRAM

A carefully designed test program is essential for the development of the waste treatment and reclamation systems. Assumptions concerning waste quantity and characteristics had to be made in many instances and it is essential that these assumptions be verified. Also, many of the recommended and alternative processes have been tested to a limited extent only on municipal-type wastewaters and little, if any, information is available on their application to wastes similar to those expected at a bare base.

The test program should include studies to characterize and minimize the quantity of waste and laboratory and pilot recommended processes. Each phase should be performed sequentially. Waste characterization studies are needed to provide information for some aspects of the laboratory studies, while the laboratory studies are needed to design and operate the pilot studies successfully.

A study is also needed to determine standards for reuse particularly with respect to organic matter, since the USPHS Drinking Water Standards do not appear applicable to reuse for human consumption.

#### 1. WASTE CHARACTERIZATION AND MINIMIZATION OF WASTE QUANTITY

It is essential that the quantity and characteristics of many of the wastes requiring treatment be determined as accurately as possible prior to the conduct of the laboratory and pilot studies. Without this information it will be impossible to design meaningful laboratory and pilot scale tests.



a. Photographic Wastes

Particular emphasis should be placed on minimizing the quantity of photographic wastewater. Reducing the quantity of this waste will be reflected in smaller incinerator and fuel requirements. Existing photographic processing units should be closely studied to determine ways of minimizing the quantity of wastewater by modifying processing techniques. The potential for reuse of washwaters, for example, requires careful consideration. Also, the reliability of the regeneration of spent ferricyanide bleaches using ozone or persulfate needs to be demonstrated in full-scale processing units. Having obtained information on possible techniques for reducing the quantity and strength of the photographic wastes, the feasible techniques should be made part of base operating procedure.

As new processes for photographic processing are developed for the U. S. Air Force, the manufacturers should be required to supply procedures for regeneration of the spent processing solutions.

A special effort should be made to determine the characteristics of the photographic washwaters apart from the concentrated process solutions. In the development of the treatment system for the combined wastes, it was decided to treat the photographic washwaters with the combined wastes. This decision was made on the basis of very limited information as to the content of these washwaters with respect to heavy metals and other contaminants. It is essential to know the composition of these washwaters in order to implement meaningful test programs.

It is essential also to have accurate characteristics of the concentrated process wastes. In particular, this fraction of the total waste should be characterized when the various solutions which will be recycled

have been removed. Incineration has been recommended for this fraction and it is important to know if the heavy metals present will volatilize at the temperatures used in incineration. The presence of such species would necessitate that they be precipitated prior to incineration.

b. Washrack Wastewaters

The available data on the characteristics of the washrack wastewaters for both vehicles and aircraft varied widely with respect to each of the various quality parameters. The reasons for these variations were many. A program to determine the washrack wastewater characteristics as precisely as possible should be undertaken.

The first step would be to determine the washing procedure for vehicles and aircraft which will be followed at base sites. Consideration should also be given to the design of washing facilities to determine whether or not all of the washwater will be collected.

If the alternate system involving treatment of the waste at the site of the washrack is followed, it is especially important to know how the waste characteristics vary as a function of time throughout the day. The necessary equalization can then be determined for uniform performance of the treatment system.

c. Combined Wastewaters

The combined wastewaters are made up of shower, lavatory, laundry, hospital, kitchen, dining room, photographic washwaters and washrack wastewaters. For purposes of this report, the characteristics were calculated based upon assumptions relating these wastes to typical municipal wastes. The USAF should undertake a testing program to determine the quantity and quality for each source contributing to the combined waste

so that an accurate assessment of the combined characteristics can be made. Individual analysis of the sources is necessary so that quantity and strength of flow can be determined as a function of time for the combined waste. This can be done by analysis of waste flows coming from similar sources at existing USAF installations.

In testing for the quantity and quality of each waste source, it is essential that an Air Force base be chosen which closely approximates the planned operation at the base. This information is necessary to develop meaningful laboratory and pilot tests.

An aerated equalization basin with a capacity of one day has been recommended for the combined treatment system. On the basis of the knowledge of the quantity and characteristics of the combined wastes vs. time for each of these waste sources, it may be possible to reduce the size of the equalization basin. Similarly, the variation in quantity and quality of the combined waste with time should be determined as a function of base size. It is possible that one day's equalization may be necessary for 1000 men whereas only one-half day's equalization may be necessary for 2000 men.

## 2. STANDARDS FOR WATER REUSE

Studies are needed to determine the desirable characteristics of reclaimed wastewater if it is to be used for human consumption. In particular, the standard for organic matter in the USPHS 1962 Drinking Water Standards of 0.2 mg/l of carbon chloroform extract (CCE) is not meaningful with respect to wastewaters which have been treated by carbon columns.

Certain compounds can pass through carbon columns; since adsorption on carbon is a key step in the CCE test, these same molecules will probably not be determined. Thus the standard for organics needs to be closely examined.

The U. S. Army Medical Research and Development Command has studies under way in conjunction with the Environmental Protection Agency to determine what standards should be used for reclaimed wastewater. These studies should be closely followed and coordinated with activities by the USAF in connection with the Bare Base Program.

### 3. LABORATORY SCALE TESTING

The means of addition of coagulant should be carefully studied in the laboratory. This can be done by studying the electrochemical means of addition of coagulant. If an iron bar can be used as a source of coagulant in place of ferric chloride, approximately one-third the weight of coagulant will have to be transported to the base.

Potential problems with electrochemical coagulant addition exist, however. The electrochemical system adds iron in the form of  $\text{Fe}^{2+}$ . Previous studies performed by the General Electric Company with electrochemical addition of iron showed that only a pin point floc was produced. Additional coagulant had to be added in order to achieve good removals of both coagulant and suspended solids.

A study of the electrochemical addition of coagulant should include the efficiency of coagulation when iron, aluminum, or a combination of iron and aluminum is used as an anode. It is essential as well that these coagulation studies be performed on bare base type wastewaters since floc

formation is influenced by the waste composition.

The flotation process should be studied using both hydrolyzing salts and electrochemical addition of coagulant. It remains to be shown that flotation will work for the candidate coagulants, at bare base wastewaters. The various operational parameters should be determined for the coagulant exhibiting most promise. Important variables to be studied are pH, total dissolved solids concentration, air to solids ratio, sludge concentration, suspended solids loading and, in particular, the type of suspended solids, i.e. emulsified oils, soap precipitates, etc. It should also be determined whether influent or recycled effluent should be pressurized with air.

In the event that flotation does not perform satisfactorily in laboratory testing, the alternative process of upflow clarification should be studied further. The solids concentration which can be achieved in the sludge blanket and the necessary operating procedures to prevent septicity in the blanket should be determined.

Laboratory studies should also be performed on activated carbon to determine design parameters for the adsorber. Carbon capacity and column breakthrough curves should be determined. The effect of using a surface loading rate of  $1.5 \text{ gpm/ft}^2$  should be studied. In addition, an important aspect related to the use of carbon is the extent to which granular carbon columns can be used for removal of suspended solids. If sufficient capacity for suspended solids removal exists, the dual media filter can be eliminated from the wastewater treatment unit. Studies should be carried out to simulate service and backwashing in order to determine 1) the ease with which solids are released from the carbon bed and 2) the rate of attrition

of the carbon when it is used in this dual capacity. There is indication in the literature that longer backwashing times are required for carbon beds as compared with sand beds if the carbon beds are used for suspended solids removal. If the rate of attrition is high, the use of a carbon bed as a filter would necessitate that additional carbon be supplied.

One of the undesirable aspects of the treatment system recommended for the combined wastes is the large amount of carbon required for the adsorber. Approximately 5300 lbs of carbon/1000-men are needed per month, assuming no regeneration. There is evidence that the carbon can be replaced in the adsorber by resins which are currently on the market. Some of these resins have very similar characteristics to activated carbon except that they can be regenerated within the adsorber using only a small amount of chemical regenerant. A given resin can be used and regenerated repeatedly, thus eliminating the demand for carbon. Initial studies should be performed on model compounds instead of on the actual wastewater in order to determine some of the characteristics of available resins. Laboratory studies using bare base type wastewaters should follow the screening studies, however.

Reverse osmosis should be studied in the laboratory to determine effluent quality which can be achieved under various operating conditions for bare base type wastes. Optimum operating conditions should be established to maintain product quality and high rates of water permeation. Effluent quality is of particular importance since this will determine whether ozonation, or another polishing process, is necessary subsequent to reverse osmosis.

In the event that reverse osmosis cannot produce the desired effluent quality, ozonation should be studied in the laboratory to determine

optimum operating conditions.

#### 4. PILOT SCALE STUDIES

The results of the laboratory scale studies should be used to determine whether the elements of the recommended system are the ones which should be utilized, or whether alternative processes should replace some of the recommended components. The pilot scale system should be designed and constructed on the basis of the laboratory scale studies. This should have the process arrangement which will be used in the bare base system. It should then be operated on bare base wastewater. The wastewater quantity and quality should fluctuate in a manner which simulates bare base operation. It is estimated that 3 to 6 months will be required for this test program.

The objective of the pilot testing is to determine 1) whether the desired effluent quality can be achieved reliably, 2) the need for process modification, 3) the desirable operating parameters and 4) design parameters for full scale operation. If process modification is required during pilot testing, sufficient time should be allotted to evaluate the modification.

## SECTION XIII

### CONCLUSIONS

#### 1. THE WASTEWATER TREATMENT SYSTEM

A wastewater treatment system is recommended for the Bare Base Mobility Program which best meets the constraints imposed by the program. It includes grinding, equalization, chemical coagulation, flotation, dual media filtration, activated carbon adsorption and chlorination and is designed to treat shower, lavatory, hospital, laundry, kitchen, dining room, pretreated washrack and photographic processing wastewaters.

It is recommended that human wastes be collected separately and incinerated in batch incinerators.

Concentrated photographic wastes, skimmings from the washrack wastes and sludge thickened by centrifugation should be incinerated.

The system has been designed in modules for 1000 men for ease in handling, assembly and dismantling.

#### 2. THE RECLAMATION SYSTEM

The recommended reclamation system consists of a reverse osmosis system, preceded by chemical conditioning of the influent to minimize membrane clogging, and followed by chlorination. The brine from the reverse osmosis process should be disposed of either by discharge to a body of water with sufficient dilution capacity, evaporation in lagoons, or by transportation to another location.



### 3. THE TEST PROGRAM

Further research is required to obtain data on the quantity and characteristics of the waste sources. Subsequent to obtaining these data, laboratory studies should be carried out to determine the suitability of the various processes which have been recommended and preliminary design parameters for each of these processes. Studies are needed on the means of chemical addition, flotation, filtration, adsorption and reverse osmosis, in particular. Following the laboratory studies, a pilot-scale test unit should be developed which will permit a 3 to 6 month study on simulated bare base wastewater to determine if the desired goals can be achieved and to determine design parameters for full scale systems.

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